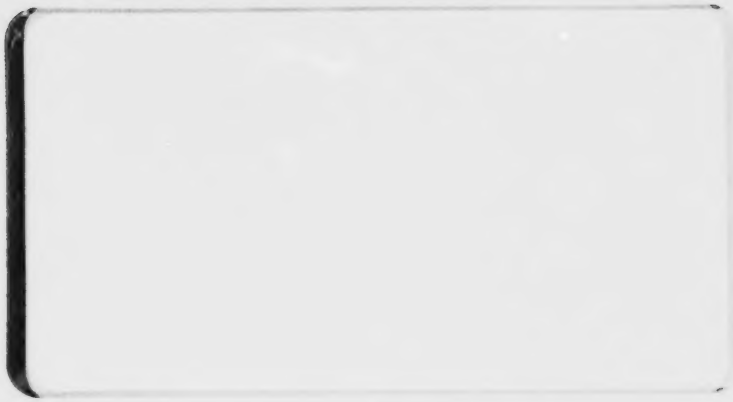


STEFFEN ROBERTSON & KIRSTEN

Geotechnical, Mining & Environmental Engineers

TECHNICAL MEMORANDUM III

METHODOLOGY FOR STABILITY AND RELEASE RATE EVALUATION

Prepared for: 2344

NATIONAL URANIUM TAILINGS PROGRAM

under DSS Contracts:
15SQ.23241-5-1709
THE TECHNOLOGY OF URANIUM TAILINGS COVERS
&
15SQ.23241-5-1715
THE STABILITY OF CONTROL STRUCTURES FOR URANIUM MILL TAILINGS

Prepared by:

Steffen Robertson & Kirsten (B.C.) Inc.
in Association with
CLIFTON ASSOCIATES LTD.
GARTNER LEE ASSOCIATES

Authors: A. MacG. Robertson
A. Ward
A. Smith
E. Watkins
D. Kent
Review: D. van Zyl
W. Clifton
R. Powell

March 1986

MICROMEDIA

PREFACE

This Technical Memorandum is the third in a series of five technical memoranda which document the data, technology and evaluation studies undertaken to evaluate the long term stability of "Covers" and "Control Structures" for Canadian Uranium Mill Tailings. The technical memoranda are:

Technical Memorandum	Title
TM I	Canadian Uranium Tailings Deposits and Structures
TM II	Characteristics Relevant to Releases of Three Canadian Uranium Tailings Sites
TM III	Methodology for Stability and Release Rate Evaluation
TM IV	Assessment of Alternative Evaluation Methods and Alternative Stabilization Practices
TM V	Evaluation of Cover and Structure Stability on Three Generic Canadian Uranium Tailings Sites

The conclusions reached and observations made as a result of these studies are summarized in two reports entitled: -

REPORT 58901 THE TECHNOLOGY OF URANIUM TAILINGS COVERS

REPORT 58902 THE STABILITY OF CONTROL STRUCTURES FOR URANIUM MILL TAILINGS

This technical memorandum reviews the technology that is available and may be employed to evaluate the stability of uranium tailings control structures and for the determination of release rates.

ACKNOWLEDGEMENTS

This study was conducted over a period of three months during which a large amount of data had to be obtained, reviewed and evaluated. It was only possible to complete the study in this short time period because of the willing and effective support provided by the Canadian uranium mining companies who were approached to provide data. Assistance was also provided in the review of the draft documents for this study by individuals employed by these mining companies.

We acknowledge with sincere appreciation the contributions of all who contributed to the study, of which the following made particularly large contributions:

Denison Mines Limited, including Mr. George Stenning and his staff and Mr. J.L. Chakravatti.

Rio Algom Limited, including Mr. A. Vivyurka and Mr. J. Maltby.

Eldor Resources Ltd., including Mr. M.P. Filion.

The staff of CANMET Laboratories in Elliot Lake, including Dr. D.G. Hedley and Dr. Nand Dave.

This study was guided by the Scientific Authority, Mr. Grant Feasby, and his prompt and effective guidance, review and comment contributed largely to its successful completion.

TABLE OF CONTENTS

		Page No.
III.1.0	**SOURCES OF TECHNOLOGY**	1-1
	III.1.1 INTRODUCTION	1-1
	III.1.2 SOURCES OF TECHNOLOGY	1-2
	III.1.3 MECHANISMS FOR LONG TERM INSTABILITY	1-4
	III.1.4 AVAILABLE TECHNOLOGY	1-8
III.2.0	**SURFACE WATER EROSION**	2-1
	III.2.1 DESIGN FLOOD ESTIMATION	2-1
	III.2.2 FLUVIAL GEOMORPHIC INFLUENCES	2-14
	III.2.3 SOIL EROSION	2-22
	III.2.4 GULLY EROSION	2-30
	III.2.5 DESIGN OF AN EROSION RESISTANT COVER	2-37
	III.2.6 CONTROL OF EROSION ON TAILINGS IMPOUNDMENT COVERS	2-37
	III.2.7 DIRECT VEGETATION TECHNOLOGY	2-42
	III.2.8 SOIL COVER	2-51
	III.2.9 WASTE ROCK	2-53
	III.2.10 WEATHERING OF ARMOURING FOR TAILINGS IMPOUNDMENTS	2-56
III.3.0	**WIND EROSION**	3-1
	III.3.1 WIND EROSION MECHANISM	3-1
	III.3.2 ESTIMATING WIND EROSION LOSSES	3-3
	III.3.3 PROTECTION OF COVER MATERIALS	3-4
	III.3.4 CHEMICAL STABILIZATION	3-5
III.4.0	**SURFACE FROST ACTION**	4-1
	III.4.1 THEORY OF FROST ACTION	4-1
	III.4.2 SURFACE FROST PENETRATION	4-3
III.5.0	**SURFACE BIOLOGICAL ACTIVITY**	5-1
	III.5.1 ROOT ACTION	5-1
	III.5.2 BURROWING INTRUSION	5-9
III.6.0	**SUBSURFACE LEACHING AND SEEPAGE**	6-1
	III.6.1 INTRODUCTION	6-1
	III.6.2 INFILTRATION THEORY	6-3
	III.6.3 EMPIRICAL INFILTRATION MODELS	6-3
	III.6.4 MEASURED PARAMETER EQUATIONS	6-5
	III.6.5 SEEPAGE PREDICTIONS	6-11
	III.6.6 FRONTAL SEEPAGE FOLLOWING ABANDONMENT	6-12
	III.6.7 LEACHING AND SOLUTE TRANSFER PROCESSES	6-16
	III.6.8 SALT MIGRATION	6-18

		Page No.
III.6.9	PRECIPITATION AND CLOGGING POTENTIAL OF UNDERDRAINAGE	6-19
III.7.0	**SLOPE STABILITY**	7-1
III.7.1	TYPES OF EMBANKMENTS	7-1
III.7.2	COMMON TYPES OF FAILURES	7-8
III.7.3	STABILITY ANALYSES	7-12
III.7.4	EVALUATION OF INSTABILITY	7-14
III.7.4.1	Likelihood of Instability and Release Rates	7-14
III.7.4.2	Material Property Changes	7-14
III.7.4.3	Geometric Changes Resulting from Erosion	7-16
III.7.4.4	Phreatic Surface Level Changes	7-17
III.7.4.5	Seismic Instability	7-17
III.8.0	**SEEPAGE EROSION**	8-1
III.8.1	INTRODUCTION	8-1
III.8.2	POTENTIAL SEEPAGE EROSION OF URANIUM TAILINGS STRUCTURES	8-1
III.8.3	SEEPAGE EROSION MECHANISMS AND ANALYSIS-UPWARD FLOW	8-1
III.8.3.1	Global Gradient Approach	8-4
III.8.3.2	Seepage Erosion through Heave	8-6
III.8.3.3	Exit Gradient Approach to Seepage Erosion	8-8
III.8.3.4	Practical Considerations with Respect to Upward Flow	8-10
III.8.4	SEEPAGE EROSION MECHANISMS AND ANALYSIS - UNCONFINED FLOW	8-10
III.8.4.1	Global Gradient Approach	8-11
III.8.4.2	Exit Gradient Approach	8-11
III.9.0	**PROBABILITY METHODS IN FAILURE COMBINATIONS**	9-1
III.9.1	PROBABILITY METHODS IN FAILURE EVALUATION	9-1
III.9.1.1	Evaluation of Single Failure Modes	9-4
III.9.1.2	Evaluation of Combined Failure Modes	9-6
III.9.2	EXAMPLES OF PROBABILISTIC ANALYSES	9-7
III.9.3	FAILURE MODES AND COMBINATIONS FOR URANIUM TAILINGS IMPOUNDMENTS	9-9

		Page No.
III.9.3.1	Cover Failure	9-14
III.9.3.2	Control Structure (Embankment) Failure	9-14
III.9.3.3	Evaluation of Probabilities	9-14
III.9.4	INTERPRETATION OF RESULTS	9-20
III.10.0	**REFERENCES**	10-1
III.11.0	**APPENDICES**	
III.2.1	CHARTS AND TABLES FOR ESTIMATION OF UNIVERSAL SOIL LOSS EQUATION PARAMETERS	A.2.1-1
III.6.1	APPLICATIONS OF DARCY'S LAW TO INFILTRATION ESTIMATION	A.6.1-1
III.6.2	RICHARDS' EQUATION	A.6.2-1
III.6.3	GREEN-AMPT PARAMETER ESTIMATION	A.6.3-1
III.6.4	GREEN-AMPT-MEIN-LARSON MODEL (GAML)	A.6.4-1
III.6.5	NUMERICAL SOLUTIONS TO THE FLOW EQUATIONS	A.6.5-1

LIST OF FIGURES

		Page No.
III.1-1	SCHEMATIC INTERACTION OF GEOCHEMISTRY AND COVER REMOVAL BY CHEMICAL ATTACK	1-12
III.2-1	HYDROLOGIC CYCLE OF STORM WATER RUNOFF	2-4
III.2-2	SOIL CONSERVATION SERVICES TYPE I AND II RAINFALL DISTRIBUTIONS	2-8
III.2-3	HUFF'S STORM DISTRIBUTIONS	2-10
III.2-4	TIME-AREA METHOD	2-13
III.2-5	IDEALIZED FLUVIAL SYSTEMS	2-15
III.2-6	RELATION BETWEEN CHANNEL SINUOSITY AND FLUME SLOPE	2-19
III.2-7	RELATION BETWEEN SEDIMENT LOAD AND FLUME SLOPE SHOWING INCREASED RATE OF SEDIMENT-TRANSPORT AT THRESHOLDS OR PATTERN CHANGE	2-19
III.2-8	RELATION BETWEEN SINUOSITY AND STREAM POWER	2-20
III.2-9	INTERACTION OF EROSION PROCESSES AT STEADY STATE ON A TYPICAL LAND PROFILE	2-23
III.2-10	CRITICAL VALLEY-FLOOR SLOPE VERSUS DRAINAGE AREA FOR SMALL BASINS, PICEANCE CREEK AREA, WESTERN COLORADO	2-31
III.2-11	CROSS-SECTION OF A TYPICAL RECLAIMED SLOPE ILLUSTRATING GULLY AND ASSOCIATED HEADCUT	2-33
III.2-12	RELATION OF STABLE SLOPE TO THE CALIBRATION CONSTANT, G	2-35
III.2-13	RELATION OF TIME-DEPENDENT SLOPE AND UNIFORMITY COEFFICIENT VERSUS OCCURRENCE OF MAXIMUM GULLY DEPTH FROM TOE-OF-SLOPE	2-36
III.2-14	RECOMMENDED PERMISSIBLE UNIT TRACTIVE FORCES FOR CANALS IN NONCOHESIVE MATERIAL	2-38
III.2-15	PERMISSIBLE UNIT TRACTIVE FORCES FOR CANALS IN COHESIVE MATERIAL AS CONVERTED FROM THE U.S.S.R. DATA ON PERMISSIBLE VELOCITIES	2-38
III.2-16	ZONES OF A RECLAIMED IMPOUNDMENT REQUIRING RIPRAP PROTECTION	2-41
III.3-1	WIND EROSION CYCLE	3-2
III.3-2	HEIGHT OF RISE VS LENGTH OF SALTATING GRAIN PATH	3-2
III.6-1	THE HYDROLOGIC CYCLE	6-2
III.6-2	GENERALIZED SOIL MOISTURE PROFILES DURING INFILTRATION AT (A) THE MOMENT OF SURFACE SATURATION AND (B) AT A LATER TIME	6-7
III.6-3	SCHEMATIC CROSS SECTION OF GROUND WATER MOUND DURING STAGE II	6-13
III.6-4	TYPICAL UNSATURATED HYDRAULIC PROPERTIES FOR A HOMOGENEOUS SOIL	6-15
III.6-5	APPROXIMATE pH VALUES (AT 25°C) AT WHICH PRECIPITATION OF VARIOUS METAL SALTS COMMENCES	6-21

		Page No.
III.7-1	EXAMPLES OF TAILINGS DAMS ON IMPERVIOUS FOUNDATIONS	7-2
III.7-2	EXAMPLES OF TAILINGS DAMS ON PERVIOUS FOUNDATIONS ILLUSTRATING SEEPAGE CONTROL	7-3
III.7-3	USE OF DRAINS ON HOMOGENEOUS CROSS-SECTIONS	7-5
III.7-4	EXAMPLES OF THIN CORE CROSS-SECTIONS	7-6
III.7-5	TAILINGS EMBANKMENT CONSTRUCTION METHODS	7-7
III.7-6	PLANAR SHEAR MODE	7-9
III.7-7(a)	ROTATIONAL SHEAR MODE	7-9
III.7-7(b)	CIRCULAR SLIP SURFACES FORMING A RETROGRESSIVE SERIES OF MASS MOVEMENTS	7-10
III.7-8	EXAMPLE OF A NON-CIRCULAR OR COMPOSITE TYPE SLIP SURFACE	7-11
III.7-9	EXAMPLE OF A WEDGE-TYPE OF SLIDING MASS	7-11
III.7-10	CROSS-SECTION SHOWING THE INSCRIBED CIRCULAR SLIP SURFACE DIVIDED INTO SLICES	7-13
III.7-11	SLOPE STABILITY CHART FOR FULLY DRAINED CONDITIONS	7-18
III.7-12	SLOPE STABILITY CHART FOR RELATIVELY HIGH WATER TABLE	7-19
III.7-13	CONTOURS OF PEAK HORIZONTAL GROUND ACCELERATION IN UNITS OF 'g' HAVING A PROBABILITY OF EXCEEDANCE OF 10% IN 50 YEARS; Za = SEISMIC ZONE BASED ON ACCELERATION	7-20
III.8-1	SCHEMATIC OF POTENTIAL SEEPAGE EROSION FAILURES IN URANIUM TAILINGS IMPOUNDMENTS	8-2
III.8-2	DEFINITION OF GLOBAL GRADIENT (i=Z/L) FOR UNCONFINED FLOW	8-12
III.8-3	SEEPAGE EROSION OF SLOPING SURFACE	8-13
III.9-1	PROBABILITY DENSITY DISTRIBUTION OF PARAMETER X	9-3
III.9-2	CAPACITY-DEMAND MODEL	9-3
III.9-3	SCHEMATIC FAULT AND EVENT TREES	9-8
III.9-4	FAULT TREE FOR EXAMPLE 1 OF LOSS OF CONTAMINATED FLUID	9-10
III.9-5	EVENT TREE FOR EVALUATION OF GEOTECHNICAL ASPECTS OF DAM SAFETY AND THE FACTORS AFFECTING PERFORMANCE	9-11
III.9-6	FAULT TREE FOR LONG TERM STABILIZATION FAILURE, BASED ON FAILURE MODES IDENTIFIED BY NELSON, ET AL (1983)	9-12
III.9-7	FAULT TREE FOR COVER FAILURE	9-16
III.9-8	FAULT TREE FOR CONTROL STRUCTURE (EMBANKMENT) FAILURE	9-17
III.9-9	FAULT TREE FOR SLOPE FAILURE OF CONTROL STRUCTURE (EMBANKMENT)	9-18
III.9-10	FAULT TREE FOR OVERALL LONGTERM STABILITY OF URANIUM TAILINGS IMPOUNDMENT	9-19

		Page No.
A.III.6.1-1	FANNING FRICTION FACTOR VS REYNOLDS NUMBER	A.6.1-2
A.III.6.2-1	GRID NETWORK FOR FINITE DIFFERENCE SCHEME	A.6.2-2
A.III.6.4-1	CONCEPTUAL MOISTURE PROFILE AT MOMENT OF SURFACE PONDING	A.6.4-2
A.III.6.4-2	GENERALIZED SOIL MOISTURE PROFILES DURING INFILTRATION AT (A) THE MOVEMENT OF SURFACE SATURATION AND (B) AT A LATER TIME	A.6.4-3
A.III.6.5-1	CUMULATIVE INFILTRATION VOLUME VS TIME	A.6.5-2

LIST OF TABLES

		Page No.
III.2-1	VALUES OF RUNOFF COEFFICIENT, C	2-6
III.2-2	RUNOFF CURVE NUMBERS FOR HYDROLOGIC SOIL-COVER COMPLEXES	2-11
III.2-3	DEFINITION OF SCS HYDROLOGIC SOIL GROUPS	2-12
III.2-4	SUMMARY OF REGIME RELATIONS FOR HYDRAULIC GEOMETRY OF RIVERS	2-17
III.2-5	CLASSIFICATION OF ALLUVIAL CHANNELS	2-21
III.2-6	EXAMPLES OF EROSION-SEDIMENT YIELD MODELS	2-26
III.2-7	PLANT GROWTH EVALUATION OF SIX TOPSOIL/OVERBURDEN SAMPLES FROM AMOK LTD., CLUFF MINE, SASKATCHEWAN AND STANDARD AGRICULTURAL SOIL TEST ANALYSIS OF THE SIX SAMPLES BEFORE EVALUATION	2-45
III.2-8	VARIATION IN PLANT GROWTH (LEGUME-GRASS MIXTURE) ON SOILS AND SUBSOILS AT 4 SITES IN EASTERN CANADA	2-45
III.5-1	WATER BALANCE SUMMARY FOR THREE ALTERNATIVE TAILINGS CAP CONSTRUCTION SYSTEMS	5-4
III.6-1	POTENTIAL GEOCHEMICAL PROCESSES FOR LEACHING OF URANIUM TAILINGS AND SOLUTE TRANSFER	6-17
III.8-1	CHARACTERIZATION OF SEEPAGE EROSION MODES	8-3
III.8-2	CREEP COEFFICIENTS AND GLOBAL GRADIENT	8-5
III.8-3	OBSERVED FLOW CONDITIONS ASSOCIATED WITH GRADIENT THROUGH TOP SOIL STRATUM	8-7
III.8-4	RECOMMENDED FACTORS OF SAFETY AGAINST SEEPAGE EROSION (EXIT GRADIENT APPROACH)	8-9
III.8-5	LIMITING GLOBAL GRADIENTS FOR UNCONFINED FLOW	8-12
III.8-6	EXPRESSIONS FOR CRITICAL EXIT GRADIENTS ON SLOPING EXIT SURFACES	8-14
III.9-1	REPRESENTATIVE COEFFICIENTS OF VARIATIONS FOR GEOTECHNICAL PARAMETERS	9-5
III.9-2	LONG TERM POTENTIAL MODES OF FAILURE	9-13
III.9-3	FAILURE MECHANISM CLASSIFICATION OF THIS STUDY	9-15
A.III.6.2-1	FINITE DIFFERENCE SCHEME DEPTH INCREMENTS	A.6.2-5

TECHNICAL MEMORANDUM NO. III: METHODOLOGY FOR STABILITY AND RELEASE RATE EVALUATION

III.1.0 SOURCES OF TECHNOLOGY

III.1.1 INTRODUCTION

During the past 15 years there has been a marked increase in the concern and interest by the public, regulatory authorities and mining industry in the environmental impact of uranium and other mine wastes. Uranium mine wastes and tailings, because of their radiological constituents, have been the focus of many of these concerns. This focus has resulted in the intense evaluation of existing mine waste disposal methods and facilities and their impact on the environment. Additional regulatory control has been implemented at both Federal and Provincial levels in most western uranium producing countries (Canada, U.S.A., Australia, South Africa and France). In some countries the State has undertaken the task of implementing the remedial measures required to clean up and stabilize abandoned uranium mill tailings.

As a consequence, considerable effort has been made by both industry and State agencies alike to develop and implement adequate stabilization and control technology. The objective of this technology is to isolate or stabilize the tailings in the depository area. Release rates of environmental contaminants must be sufficiently low to satisfy release criteria which ensure that an acceptable level of environmental impact is achieved in both the short and the long term.

During the operating period, while tailings are being produced, manpower and equipment resources are available at the mine site to monitor the release rates. If release rates exceed specified limits, remedial action can be implemented. Following closure of the mill facilities, the tailings deposits remain as a long term potential source of radioactive materials and other environmental contaminants. The rate at which these contaminants are released to the environment will change with time as: -

(i) the tailings change their chemical and physical nature as a result of the weathering and other hydrogeochemical processes that persist, and

(ii) the containment control structures are subject to similar physical and chemical processes which modify their ability to function in the manner in which they were originally conceived and designed.

If the rates of change are very, very small then the changes in the release rates may not be significantly different for a very long time. Small but perpetual change which results in cumulative increases in release rates may bring about unacceptable rates of releases in lesser time periods. Rapid changes could result in catastrophic releases in relatively short periods.

Critical to an evaluation of the long term environmental impact of uranium tailings is the stability of both the tailings products and the containment structures. Stability is that characteristic of a material or structure which indicates its resistance to change. A structure or material is

stable if, over the period of interest, the change in it is sufficiently small not to make a material difference to the rate of the contaminant releases with which it is associated. Conversely, it is unstable if the release rate changes materially or becomes unacceptably high.

The tailings materials themselves are metastable in that they have been crushed and placed in an environment which differs considerably from that existent at the location of the original ore. Considerable change may occur in the physical and chemical nature of the tailings before they reach equilibrium under the conditions of this new environment.

Most of the physical and chemical change of concern to the long term stability of uranium tailings and their containment structures are relatively slow but progressive. It is necessary to know both the rate of change and the time period of interest before an assessment can be made of the long term stability of the materials and structures.

The period for which the tailings will remain as potential contaminant sources exceeds by a number of orders of magnitude the periods for which man can economically design and construct tailings containment structures. Thus the period of interest is determined by that period over which current design and construction technology can be applied with some confidence.

At the onset of this study it was presumed that current design and construction technology should be able to achieve stable structures over periods of at least 100 to 200 years. Since the purpose of this study is to evaluate the long term stability of such structures, an evaluation period extending to between 1000 and 2000 years into the future was selected.

This study is directed to an evaluation of the long term stability of containment structures that surround and cover the tailings impoundment. Separate studies by Golder Associates, 1984, and Clifton Associates, 1986, have been completed on behalf of the National Uranium Tailings Program to evaluate the stability of liners which may be used to contain seepage from uranium tailings deposits.

III.1.2 SOURCES OF TECHNOLOGY

Relevant technology has been developed first in those locations where need and concerns were most pressing.

Uranium milling has been done in both the U.S.A. and Canada since the early 1940's. In the U.S.A. a greater geographical diversity of sites were involved, many of which were operated for comparatively short periods before the mills were closed. A number of these abandoned tailings facilities were located in or adjacent to large population centres in which industries other than uranium mining were the primary basis for the economy. Salt Lake City in Utah and Grand Junction in Colorado are examples. Dissemination of the tailings, with the associated radionuclides, into the surrounding inhabited areas, through its use as a building material for example, resulted in an early awareness and concern regarding the environmental impact and health effects. In November, 1978, the United States Congress enacted the Uranium Mill Tailings Radiation

Control Act which mandates that the Department of Energy (DOE) implement remedial action programs at designated inactive uranium mill tailings sites (Matthews, 1981). Twenty-five tailings sites were designated in 1979. Since then, both the U. S. Nuclear Regulatory Commission (NRC), which is responsible for the licensing and control of active uranium mill tailings sites (in Non-Agreement States), and DOE, through their Uranium Mill Tailings Remedial Action Program (UMTRAP) have been very actively sponsoring and conducting research into the technology of uranium mill tailings stabilization (O'Brien, 1981).

The NRC and DOE have sponsored a large amount of research on a wide variety of relevant topics. Much of this work has been undertaken at the Pacific Northwest, Oakridge, Los Alamos, Sandia and Argonne National Laboratories, as well as a number of universities and private contractors. The Colorado State University, in particular, has done considerable work on the design and evaluation of long term stability. Much of this work is published in reports in the public domain prepared for the NRC and DOE or published in the Proceedings of the Symposium on Uranium Mill Tailings held annually at the Colorado State University in Fort Collins, Colorado.

The containment structures that are proposed are large and it requires a considerable period of time to develop and evaluate appropriate technology, and to construct and execute appropriate laboratory or field tests. Many of the principles can only be evaluated in full scale field applications and on time scales approaching those for which the facilities are designed. In the USA, DOE is required to implement the remedial actions at the designated sites within seven years of issuance of EPA standards in January, 1983. The implementation of the first UMTRAP designed abandonment schemes are now being completed (Matthews, 1986) and field data on actual costs and the performance of the structures designed and constructed using the developed technology is beginning to become available. A review has been made of waste management programs at operating uranium recovery facilities in the U.S.A. as they relate to the currently promulgated standards and is reported by Gillen et al, 1985.

In Canada the new awareness of the effects of uranium mill tailings and the requirement for a new level of tailings control technology developed substantially as a result of the Cluff Lake Board of Inquiry (Bayda et al, 1978), which set guidelines for the industry. Both the Canadian Atomic Energy Control Board (AECB) and the Department of Energy Mines and Resources (CANMET) considerably increased their programs of research into the development of appropriate tailings control technology. A large (probably the largest) contribution to technology development was made by the uranium mining and milling industry by individual uranium deposit developers in their response to the increased public concern and to the new guidelines for uranium tailings control technology issued by the AECB, Saskatchewan and Ontario Provincial Authorities. Some of this technology was based on research done elsewhere but much was developed in response to the conditions and concerns specific to the Canadian sites. The results of this research are available through the AECB, CANMET, Provincial Authorities (such as Saskatchewan Environment) and scattered in the proceedings of symposia (such as that of the University of Saskatchewan 1985).

In September, 1982, the Minister of State for Mines announced funding approval for the Canadian National Uranium Tailings Program (NUTP) as part of CANMET. Funding of research through this program has enabled the scattered Canadian research to be brought together and wed as a basis for further development. Previous studies sponsored by NUTP are of direct relevance to this study. The results of this program are of large consequence to the Canadian environment and uranium mining industry since it is focused on Canadian conditions and appropriate technology often differs materially from that being developed in the U.S.A. and elsewhere.

Research continues also in other countries on the stabilization of uranium mill tailings. An early study by the International Atomic Energy Association (IAEA) resulted in their published technical report No. 209 (IAEA, 1981). The Nuclear Energy Agency (NEA) has had an ongoing program on the topic since 1978 (Rafferty, 1981). Most of the research done for these agencies is done by the individual member country organizations. Results are available through reports issued by the national member country organizations and in NEA and IAEA sponsored symposia. Much of the most advanced and relevant work is being done in North America through the agencies previously mentioned.

There are a large number of areas where related technology is of direct relevance to the long term stabilization of uranium tailings deposits. Of particular relevance is the work done on sedimentation and erosion on mining disturbed lands at the University of Kentucky (Ward et al, 1979a), hydrogeochemical contaminant migration at the University of Waterloo and seepage modelling at the University of Saskatchewan. The technology of flood prediction and geotechnical engineering design and construction practices is extensively developed in other areas of engineering application.

III.1.3 MECHANISMS FOR LONG TERM INSTABILITY

A uranium tailings impoundment is considered unstable in the long term if unacceptable rates of release of contaminated solids and liquids occur to the environment. A stable reclamation scheme is therefore required to not allow such releases over the 1,000 to 2,000 year period.

Release of solids can occur due to a failure of the reclamation cover and/or a failure of the containment structure. Such failures can be brought about by long term degradation or by large short term force, e.g, high runoff eroding the cover and/or embankment. Long term degradation refers to changes in the original design conditions. For example, the design assumes efficient operation of the underdrains and a low phreatic surface; due to long term mechanisms the drains clog and the phreatic surface rises leading to slope instability through mass failure or seepage erosion failures.

The mechanisms for long term instability are divided into two groups, namely surface mechanisms and sub-surface mechanisms.

A. Surface Mechanisms

(i) Erosion. Three forms of erosion are of concern:

(a) Sheet erosion. This is caused mainly by wind erosion and raindrop effects. Standard methods are available for the assessment of freshly disturbed surfaces. Rates applicable to old surfaces are determined by the degree of crusting, natural surface armouring and vegetation that develops. The Universal Soil Loss Equations (USLE) conventionally used for the estimation of such erosion rates are generally un-realistic for the very long term predictions required under Canadian conditions. The USLE can be used as a basis but parameters must be greatly modified (based on experience) if it is to be a realistic tool for Canadian conditions.

(b) Rill erosion. This is one of the most important causes of instability. The potential is dependent on surface cover and slope, erosive characteristics of soil, preferential flow paths (such as animal tracks down embankments), catchment, run-off and precipitation characteristics. Research has just been completed on this topic at Colorado State University (Falk et al 1985). Using this work as a basis, appropriate rill erosion evaluation and control measures can be developed for Canadian conditions.

(c) Flood erosion. This occurs where a significant drainage crosses or is adjacent to the tailings impoundment which, during flood events, could scour the structure. The adoption of the maximum probable flood is a conservative approach. However, the use of probabilistic methods allows evaluation of less extreme values. The potential for scour, given a particular flood event, can be assessed from available technology. The geomorphology of stream and river development must be considered where this could endanger an impoundment. Where impoundments contain or store water, the risk of overtopping or of blockage (by ice, slope failure, vegetation or debris) must be considered.

(ii) Biological Effects

(a) Intrusion by both man and animals can result in physical disruption of both the covers and embankment structures. This can bring tailings to the surface and increase infiltration.

(b) Vegetation has a beneficial effect in that it prevents erosion. Root action may produce perforations through low permeability covers or cause clogging of coarse suction barriers. Increased infiltration and groundwater mounding may result. Vegetation development or loss promotes frost aggradation and degradation with attendant concerns discussed below.

(c) Bacterial action in the underdrains has been responsible for longterm drain plugging in sulphide rich ores.

B. Sub-Surface Mechanisms

(i) Frost Action

Frost action consists of seasonal freezing and intermittent permafrost. Long term weather changes, while producing only small average temperature variations may result in large differences in the potential for intermittent permafrost. Changes on a 200 to 2000 year scale must be considered.

Seasonal frost, particularly in the very severe winters in a cold cycle, may penetrate deeper than provided for in the original designs. Underdrain system exits and embankment faces may freeze, blocking drainage. This increases the potential for piping and mass slope failures.

Frost aggradation could occur to such an extent that pingos may develop, resulting in frost boils which may lift tailings through covers to the surface. They could also cause cracking and induce piping failure.

Rapid permafrost degradation releases pore water which cannot drain sufficiently fast in fine grained soils, and can result in pore pressure generation. This causes slope sloughing. Such rapid aggradation and degradation would be anticipated where fine grained cover materials (which are frost susceptible) are used to control infiltration. Vegetation growth would promote aggradation and, if this is suddenly lost, (due to a fire, drought or lumbering activities), then rapid aggradation could occur.

(ii) Increase in Phreatic Surface

All slopes should be stable at the time of impoundment abandonment with the "then-existing" phreatic surface. The position of this surface may change with time. If the phreatic surface increases in level, then there is an increased potential for failure. Some reasons for long term increases would be:

- Increased precipitation due to long term climatic changes or a series of "wet" years;
- Increased infiltration due to cover loss or increased cover permeability or ponding of water in depressions on the cover;
- Loss of effectiveness of liners, i.e. degradation of synthetic liners or piping of bentonite/sand liners; and
- Loss of effectiveness of underdrainage systems due to chemical precipitation, physical piping of fines, frost action, roots or bacteria activity.

(iii) Plugging of Underdrains

Underdrains may be engineered, such as at Key Lake, or drainage may exit by seepage through an impoundment embankment. Decreases in the permeability of these drains occurs if the drain material weathers, if exsolution from seepage fluids forms precipitates in the drainage material pore spaces or if freezing occurs.

Geochemical precipitation and deposition is highly probable, particularly where sulphide-bearing uranium tailings are deposited - for example, iron sulphides at Elliot Lake or nickel arsenic sulphides in some Saskatchewan deposits. Acidification, due to sulphide oxidation from infiltration leaching of the tailings, results in solution and mobilization of a number of chemical species, such as iron, calcium, sulphate and silicon. Once the resulting solution exits to the often partially saturated underdrain system, the geochemical environment alters which changes the solubility of many of the species held in solution. Combining such an environment change with an increase in pH value or alteration of redox potential will result in precipitation or formation of a number of only partially soluble species.

Among precipitates which can form are:

- iron minerals
  - jarosite (iron, aluminum sulphate)
  - siderite (iron carbonate)
  - melanterite and halotrichite (iron sulphate/hydroxides) see Smith and van Zyl, 1983.
- gypsum (calcium sulphate)
- silica

If such precipitation occurs in the intergranular spaces of the underdrains or along any seepage path, permeability changes will result in partial or complete blockage. Such blockage can be rapid (a few years or tens of years) and such problems have been experienced (gypsum and iron salts precipitation) in several operating and decommissioned (non-uranium) tailings impoundments.

Physical piping of fines into the underdrains could occur over a long period of time due to initial construction imperfections. It could also occur suddenly due to degradation of filter layers (by root, rodent or other animal activity) or degradation of an overlying liner and increase in the seepage gradient. The integrity of synthetic liners for 200 to 2000 year terms is highly questionable and it is anticipated that some of the presently-installed, bentonite /sand liners do not meet filter criteria relative to underlying materials.

Frost, particularly seasonal freezing, is a common cause of drain plugging.

(iv) Material Degradation

The weathering of embankment materials with consequential losses in material strengths is not considered to be a significant contributor to long term stability. Of concern is deterioration of some of the other elements, namely:

- degradation of rip-rap (Walters 1982, Lindsey et al, 1982);
- degradation of vegetation cover due to chemical and physical changes to the cover (salt migration, settlements causing ponding, erosion) and growth environment; and
- degradation of underdrain materials.

III.1.4 AVAILABLE TECHNOLOGY

A. Covers

A review of available technology for covers is provided by Beedlow and Parker (1985). Guidelines and manuals that have been developed are based on sound technology for short term (current or a few years) solutions with modifications to allow for extrapolations or projections of long term conditions or performance. Short term conditions are climate, ore and site specifics so that sound technology for one area (a hot arid zone for example) may not be relevant to another (a wet cold zone for example). The applicability of developed guidelines to Canadian conditions and to different regions and ore types within Canada requires an evaluation of the differences that exist. A significant amount of research has already been performed on topics such as vegetation covers, salts migration and physical stability of covers in Canadian conditions. This data has been reviewed together with available data on Canadian tailings deposits (particularly site characterization data developed by the NUTP program) for the identification of technology appropriate to the various ore types and mining regions in Canada.

B. Structures

Long term processes are not well understood and a research program is currently being completed at the Colorado State University (CSU) for the U.S. NRC on the long term stability of uranium retaining structures. This research, while specific to U.S. deposit conditions, forms a basis from which to initiate the evaluation for this study program. This ensures that the most current compilation of international literatures and evaluations are available at the time of study initiation.

C. Alternative Cover Technology

Alternative cover materials that may be considered range from direct vegetation through various thicknesses and types of soil and armoured rip-rap to water cover.

(i) Soil and rip-rap covers

Design guidelines have been developed for soil and rip-rap covers, largely to ensure containment of radon flux and to resist erosion. A "Radon Attenuation Handbook for Uranium Mill Tailings Cover Design" (Rogers et al, 1984) has been published by the NRC which is considered adequate for design in Canada. Of concern is the potential disruption of the cover and defects in the covers (Mayer and Zimmerman, 1981).

To prevent erosion by wind and water, soil covers may be protected by vegetation covers or rip-rap. Rip-rap is favoured in the more arid areas of the U.S.A. and considerable work has been done on the suitability of rip-rap and its long term survivability as described in Lindsey et al, 1982, Walters, 1982; Foley et al, 1985, and Ecker, 1984. An NRC program of continued research on rip-rap is currently being done at Colorado State University.

Long term erosion of covers have been evaluated and computational methods described by Walters, 1983 and Walters and Skaggs, 1986. Attempts have been made to evaluate the reliability of these estimation methods by evaluating the survivability of ancient man-made earthen mounds (Lindsey et al, 1983). Little confidence exists in the applicability of these methods to Canadian conditions. The low erosion rates occurring in the uranium mining areas is demonstrated by the low suspended solids in the abundant surficial waters. Records of suspended solids may well provide a suitable basis for the determination of erosion parameters for Canadian conditions. A re-evaluation of these methods, for Canadian conditions, is essential.

Erosion by wind can be an active erosion mechanism as is currently evident at the Gunnar tailings site. Classical wind erosion methods can be used to estimate deflation rates and studies have been done on its control for uranium tailings (Li et al, 1983 and Elmore and Hartley, 1984). Any long term solution must achieve stability against wind erosion, and wind erosion deflation rates are therefore not of direct relevance to this project.

The integrity of soil and rip-rap covers can be disrupted as a result of:

- consolidation of the underlying tailings or slimes to the extent that cracking occurs in the cover
- rill erosion
- burrowing animals and insects
- vegetation root penetration

These aspects have and are being considered in studies such as those reported in Voorhees et al, 1983, and Nelson et al, 1978 and 1983.

(ii) Vegetation covers

Design guidelines have been prepared by Beedlow (1984) but are specific to U.S. tailings. They are not directly applicable to Canadian conditions. Vegetation cover has proved to be an extremely effective stabilizing agent on many tailings facilities. Its effectiveness in stabilizing tailings spills at Beaverlodge is clearly demonstrated by the evaluation studies made for the close-out of that mine. The success of establishing vegetation directly onto tailings is dependent on the growth environment. These vary drastically from tailings site to tailings site and on different areas of the same tailings deposit. Conditions such as drainage, available nutrients and acid generation, effect both the success of establishing the vegetation and the succession of plant growth that will develop. Substantial modification of the plant growth conditions and careful selection of the plant species and initial care is required to successfully develop and maintain a plant cover.

The technology of vegetation cover establishment is complex and design guidelines are useful only to establish fundamental requirements. Detailed implementation often requires site and tailings specific research and testing to develop a successful re-vegetation program.

(iii) Water cover

The Canadian uranium mining areas are unique regarding the large quantities of surface water and lakes. Studies such as those undertaken for the tailings disposal in Rabbit Lake open pit (Gulf Minerals, 1981) and the close-out study at Beaverlodge (Eldorado Resources, 1983), have demonstrated the effectiveness of water covers. This alternative has not been considered in previous evaluations (Beedlow and Parker, 1985) but is a reasonable option for Canada. Seepage by convection, advection, and dispersion are important considerations for this cover type.

D. Alternative Cover Materials

The material types that are locally available determine the suitability and cost effectiveness of different cover materials.

Water is generally freely available but the implementation of a water cover depends on the ability to locate a water cover above a deposit without developing large seepage gradients through the tailings deposit. It is therefore potentially suitable for future deposits but may not be applicable to many existing deposits.

Vegetation directly on the tailings does not sufficiently limit radon and gamma for it to satisfy requirements for the more radiologically active uranium ores. On low activity tailings in remote deposits and in conjunction with earth covers it has a very important role. Vegetation type must be selected to suit the soil/tailings type, climate and moisture. The effects of succession and compatibility with other environmental requirements are also important. A wide variety of alternative approaches

can be taken but it will generally be possible to establish a vegetation cover. The survivability of the cover subject to chemical changes and salt migration in the tailings and covers must be allowed for.

Soil and rip-rap covers depend on available local materials. Canadian uranium deposits are located almost entirely in the zone of recent continental glaciation. The soils encountered in the vicinity of the tailings deposits are therefore young soils, derived from glacial deposits and outwash. Comparatively little weathering and decomposition of the soils have occurred. Thus they consist largely of glacially produced till, sand, silt with clay sized rock flour but little true clay. They are chemically relatively inactive. There is generally little very low permeability clay available. Tills are often the best source of fairly low to low permeability construction materials. At Key Lake, use was made of imported bentonite to manufacture the desired low permeability impoundment liner layers.

E. Geochemical Considerations

In order to determine the long term performance of a cover over deposited tailings, there is a requirement to understand and interpret the geochemistry of the cover material/tailings interactive system. While chemical reactions may take place in the cover material and the tailings apparently independently, the movement of reaction products in solution either by seepage downwards or capillary action upwards due to evaporation/evapotranspiration results in an interactive and mutually dependent system.

Figure III.1-1 illustrates schematically this interdependence. For example, as rainfall/snowmelt infiltrates the cover material, there may be solution reactions and leaching in the cover. The reaction products are carried downward into the tailings. The relatively oxygen-rich infiltrate may then reach the tailings, and possibly, cause acid generation due to sulphide or iron oxidation in the tailings. Subsequently, due to the capillary forces induced by evaporation, the acidic interstitial flow in the tailings, which will be high in dissolved solids also, will move upwards, resulting in chemical attack on the cover material. Such chemical attack will degrade the cover material in terms of its permeability, capacity for sustaining vegetational growth and erodibility. The net result may be loss of cover, further infiltration and chemical reaction, followed by complete failure of the cover.

It is important, therefore, in any study of cover material performance, to identify and understand the complete geochemistry of the system as well as identifying the location and nature of specific reactions within such a system. Such reactions may include:

- initial solution reactions in the cover, e.g., loss of soluble carbonates, such as calcium carbonate;

- degradation of mineral species due to weathering, e.g., olivine, pyroxene, feldspars, etc. to clays and release of magnesium, iron, etc;

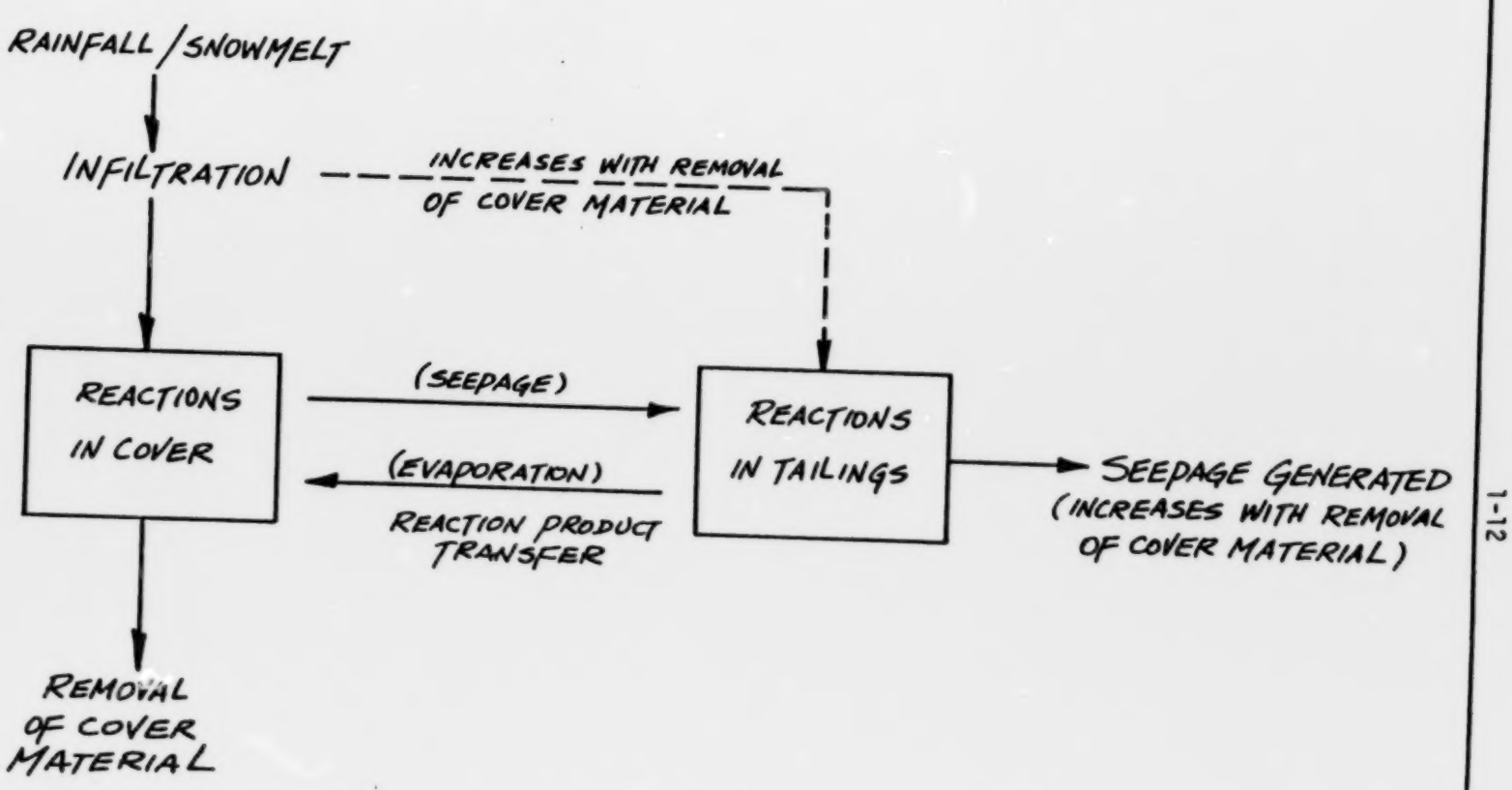


FIGURE III.1-1

SCHEMATIC INTERACTION OF GEOCHEMISTRY AND COVER REMOVAL BY CHEMICAL ATTACK

- precipitation of metasoluble species from the interstitial liquids in both the cover material and the tailings, for example, iron hydroxides and calcium sulphate (gypsum);
- acid-generation in the tailings from either sulphide oxidation or ferrous to ferric iron oxidation;
- secondary solution of trace metal species due to acidification;
- possible oxidation of residual sulphide minerals in the cover material; and
- acid solution of mineral species in the cover material due to transfer of interstitial liquid upwards from the oxidizing tailings.

This list is not exhaustive, but illustrates the type of considerations necessary. Any consideration, however, must be based on a fundamental appreciation of the chemistry and mineralogy of the cover material and the tailings. With uranium tailings in Canada, there are semi-distinct chemical types, the three main groups of which are:

- high sulphide/iron, acid generating tailings (e.g. Elliot Lake);
- high nickel/arsenic sulphide tailings (e.g., Key Lake);
- "clean", non-sulphide bearing tailings (e.g., Cluff Lake/Rabbit Lake).

In addition, while all these types of tailings are normally neutralized prior to discharge, there are differences in ore processing which account for changes in the final tailings composition: for example, the use of sodium chloride or ammonia in the process will result in elevated levels of these species, or their oxidation products in the tailings.

F. Seepage Considerations

Water passing through the cover (or upper tailings surface) becomes contaminated and leaves the tailings area as contaminated seepage. The long term seepage from the impoundment must be equal to or greater than this rate of infiltration. The importance of infiltration control is demonstrated by this and the observations in the previous section.

III.2.0 SURFACE WATER EROSION

III.2.1 DESIGN FLOOD ESTIMATION

Generally, dams, runoff conveyance systems, flood control systems, earthworks and waste disposal systems are designed to control a prescribed design storm event. Commonly used events are the 1 in 10 year event, the 1 in 100 year event and the Probable Maximum Flood (PMF). On the average a 1 in 10 year event might be expected to occur once every 10 years. However, in the short term there might be several 1 in 10 year events in a 10 year period and a chance that there is more than one 1 in 10 year event in a single year.

It should also be noted that the term 'design storm event' is applied to both storm rainfall and storm runoff. It should not be expected that the 1 in 10 year rainfall event will produce the 1 in 10 year flood event. Storm runoff is very dependent on antecedent moisture conditions. A severe event following very dry conditions may produce only a small amount of runoff, while a small event following a wet period could produce a large volume of runoff. It is also necessary to assign a rainfall duration to the return period of a rainfall event. For example, the 1 in 100 year, 1-hour rainfall depth might be 50 mm and the 1 in 100 year, 24-hour rainfall depth 120 mm.

The PMF is defined as the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The rainfall condition associated with the PMF is known as the Probable Maximum Precipitation (PMP) event and is defined as the theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage basin at a particular time of year. The PMF design event is the most stringent hydrologic design criteria which can be practically applied and is only used if the consequences of failure might be catastrophic or if the design life of the structure is long (at least 100 years). Recently Nelson et al., (1983) suggested that the design event for evaluating the long-term stability of a reclaimed tailings impoundment should be the PMF.

Prior to looking at techniques for making design flood estimations it will be beneficial to first consider the importance of selecting an appropriate design return period. The probability of at least one exceedance of a design event is given by:

$$f(pt,n) = 1 - (1-1/T)^n \qquad (2.1)$$

where T is the design return period, pt is equal to 1/T, and f(pt,n) is the probability that a T-year event will be equaled or exceeded at least once in an n-year period. If n is equal to T then f(pt,T) approaches 0.63. This means that if a structure with a design life of 100 years is designed based on the 1 in 100 year event the probability is about 0.63 that the design capacity will be exceeded at least once during the design life. If the same structure was designed based on the 1 in 200 year event there would still be the very high probability of 0.33 that the design capacity would be exceeded at least once during the design life. A design return

period in excess of 1 in 1000 years would need to be used to provide a 95 percent confidence that the design condition is not exceeded.

Design Flow Determination

Flood flows associated with a particular return period are either determined from streamflow records or calculated based on rainfall information and knowledge of the flood flow characteristics of the catchment. In determining design flows for tailings impoundments, estimates will usually be made based on rainfall and catchment information. Although it might be necessary to divert small streams, tailings impoundments are seldom located in the flood plains of major rivers. However, regional relationships between streamflow rates and catchment characteristics can provide a very useful means of determining design flood flows.

Although not strictly correct, it is generally assumed that flood flows for a given return period will be produced by storm events with the same return period. Therefore, the design hydrologist will use the rainfall depth associated with a 1 in 100 year rainfall event to determine the 1 in 100 year peak flood flow. The PMF is determined based on the PMP and if high runoff conditions are assumed (ie wet antecedent moisture conditions or frozen ground) errors in associating the PMF to the PMP will generally be small. However, it should be noted that in regions where peak flows are caused by snow melts it is not appropriate to make flood estimations based on storm rainfall information.

The long term stability of a tailings impoundment could be affected by:

1) Flows from an upstream catchment which might impact on the toe or face of the impoundment;

2) Flows associated with precipitation falling on the sides of the impoundment; and

3) Flows associated with precipitation falling on the top of the impoundment.

The severity of each of these impacts will be dependent upon on-site and regional conditions. It can be seen that on-site, local and regional flood flow determinations might need to be made as part of a long-term stability analysis for a tailings impoundment.

In making flood estimations Barfield et al., (1981) noted that there are 5 cases that a design hydrologist might be faced with, namely:

1) A reasonably long record of streamflow is available at or near the point of interest on the stream of interest;

2) A reasonably long record of streamflow is available on the stream of interest, but at a point somewhat removed from the location of interest;

3) A short streamflow record is available on the stream of interest;

4) No records are available on the stream of interest, but records are available on nearby streams;

5) No streamflow records are available in the vicinity

Streamflow records and surface water data for Canada is published on an annual basis by the Water Resources Branch of the Department of the Environment in Ottawa. Additional information is presented in the Hydrological Atlas of Canada (Minister of Supply and Services, 1978). Procedures for determining flood flows for Cases 1 to 4 are standard and can be found in any hydrology text such as Barfield et al, (1981).

Case 5 flood flow determinations are made empirically based on rainfall and catchment information. Design rainfall depths for different return periods are presented in the Hydrological Atlas of Canada. These maps have been developed by preparing depth duration frequency curves for different weather stations throughout the country. Generally, historical rainfall records are only available for periods of less than 100 years and it is necessary to extrapolate this information to make PMP estimates. Point information and maximum storm information from adjacent catchments is often transposed to obtain areal PMP estimates.

Runoff associated with rainfall is a function of a number of complex interactions on a watershed. This process is known as the hydrologic cycle. For the purpose of making flood estimations, the cycle is modelled as the fairly simple system illustrated in Figure III.2-1. The following components of the cycle need to be modelled:

1) rainfall

2) surface storage and detention

3) infiltration

4) interflow

5) channel streamflow

The manner and degree to which these factors are accounted for varies from one flood flow technique to another.

A number of flood flow estimation techniques have been developed. The most commonly used approaches are:

1) empirical equations

2) unit hydrograph techniques

3) time-area method

4) kinematic method

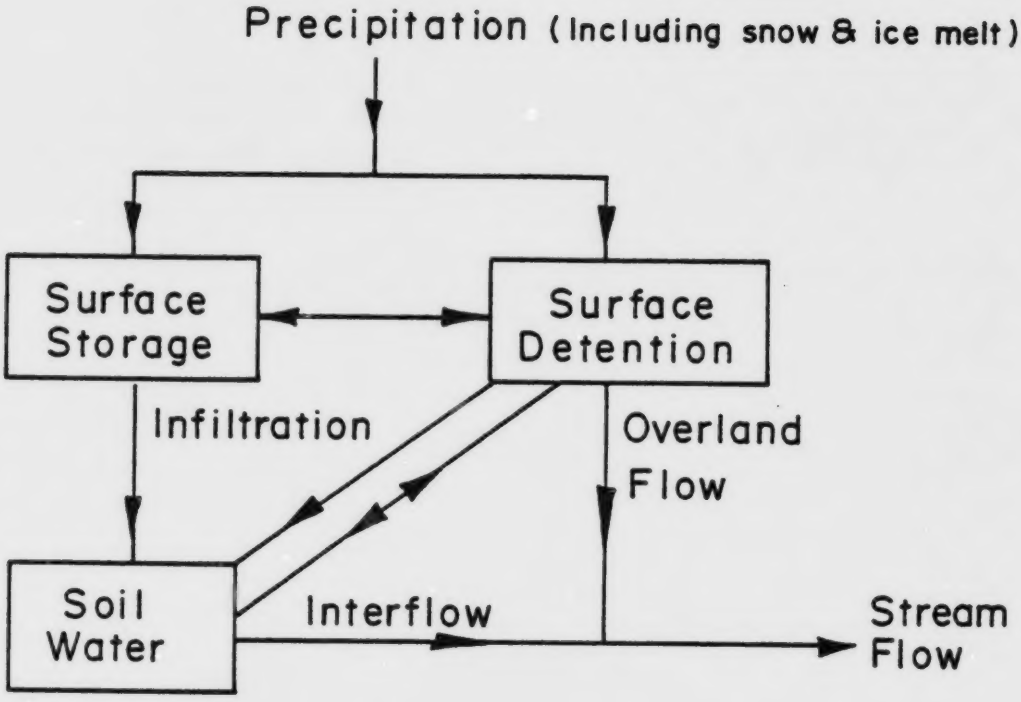


FIGURE III.2-1 HYDROLOGIC CYCLE OF STORM WATER RUNOFF

Numerous models have been developed, based on these approaches, and only the more commonly used techniques will be discussed.

Empirical Equations

The Rational method is by far the most widely used of the empirical equations, with the equation being:

$$Q = CiA \qquad (2.2)$$

where Q is the peak flow , C an empirical coefficient, i the average rainfall intensity during the time of concentration, and A the catchment area. The empirical coefficient C can be determined from the information presented in Table III.2-1.

This method while being the oldest and simplest of the available methods, has spawned numerous ways of computing the coefficient "C" and the time of concentration. The method has many limitations and can only be used to determine peak runoff rates.

The method is based on the following assumptions:

1) The rainfall occurs uniformly over the drainage area

2) The peak rate of runoff can be reflected by the rainfall intensity averaged over a time period equal to the time of concentration of the drainage area

3) The time of concentration is the time required for flow to reach the point in question from the hydraulically most remote point in the drainage area

4) The frequency of runoff is the same as the frequency of the rainfall used in the equation.

The Unit Hydrograph Approach

A unit hydrograph is a hydrograph of runoff resulting from a unit of rainfall excess occurring at a uniform rate, uniformly distributed over a watershed in a specified duration of time. The approach is empirical and based on the following assumptions:

1) Uniform distribution of rainfall excess over the watershed

2) Uniform rainfall excess rate

3) The runoff rate is proportional to the runoff volume for a rainfall excess of a given duration

Reference should be made to any standard hydrology text for details on the development of a unit hydrograph. The three basic factors which need to be determined in order to develop a unit hydrograph are the time to peak of the unit hydrograph, its peak flow rate and the shape of the hydrograph.

TABLE III.2-1 VALUES OF RUNOFF COEFFICIENT,C

Character of Surface	Runoff Coefficients	
	Range	Recommended
Pavement--asphalt or concrete	0.70-0.95	0.90
Gravel, from clean and loose to clayey and compact	0.25-0.70	0.50
Roofs	0.70-0.95	0.90
Lawns (irrigated) sandy soil		
Flat, 2 percent	0.05-0.15	0.10
Average, 2 to 7 percent	0.15-0.20	0.17
Steep, 7 percent or more	0.20-0.30	0.25
Lawns (irrigated) heavy soil		
Flat, 2 percent	0.13-0.17	0.15
Average, 2 to 7 percent	0.18-0.22	0.20
Steep, 7 percent	0.25-0.35	0.30
Pasture and non-irrigated lawns		
Sand		
Bare	0.15-0.50	0.30
Light vegetation	0.10-0.40	0.25
Loam		
Bare	0.20-0.60	0.40
Light vegetation	0.10-0.45	0.30
Clay		
Bare	0.30-0.75	0.50
Light vegetation	0.20-0.60	0.40
Composite areas		
Urban		
Single-family, 4-6 units/acre	0.25-0.50	0.40
Multi-family, >6 units/acre	0.50-0.75	0.60
Rural (mostly non-irrigated lawn area)		
<1/2 acre - 1 acre	0.20-0.50	0.35
1 acre - 3 acres	0.15-0.50	0.30
Industrial		
Light	0.50-0.80	0.65
Heavy	0.60-0.90	0.75
Business		
Downtown	0.70-0.95	0.85
Neighborhood	0.50-0.70	0.60
Parks	0.10-0.40	0.20
Rural open space		

(ASCE, 1970, and Seelye, 1960)

The time to peak is determined as:

$$t_p = t_L + D/2 \qquad (2.3)$$

where t_L is the lag time and D is the time increment of the rainfall excess. The lag time can be estimated as (USDA, 1973):

$$t_L = 0.6\, t_c \qquad (2.4)$$

where t_c is the time of concentration.

Numerous equations have been developed for determining the time of concentration and lag time. Commonly used methods include that of Kirpich (1940), the SCS (1972 and 1975), Overton (1971), Overton and Crosby (1979) and Ward et al. (1981). In addition to these methods several regional regression methods have been developed.

Perhaps the most commonly used methods are those developed by the SCS. The SCS lag equation is:

$$t_L = \frac{L^{0.8}\,(S + 1)^{0.7}}{1900\, Y^{0.5}} \qquad (2.5)$$

where t_L is the lag time in hours, L is the hydraulic length of the watershed in feet, S is a function of the SCS curve number and Y is the average land slope in percent. The peak flow of the unit hydrograph can be estimated by the SCS equation (SCS, 1972):

$$q_p = 484\, A/t_p \qquad (2.6)$$

where q_p is the peak flow in cfs, A is the watershed area in square miles and t_p is the time to peak in hours. The shape of the unit hydrograph is usually modelled as:

1) a single triangle

2) curvilinear

3) a double triangle

The SCS has developed single triangle and curvilinear unit hydrograph procedures (SCS, 1972). These are the most commonly used methods. Double triangle shapes have been proposed by Overton and Troxler (1978) and Ward et al, (1981). To develop a storm hydrograph based on unit hydrograph procedures it is necessary to obtain a rainfall excess hyetograph. Generally, the design storm rainfall depth is associated with a synthetic rainfall pattern and abstractions from the rainfall are estimated by using an empirical procedure.

The most commonly used synthetic rainfall procedures are the SCS Type 1 and 2 patterns (SCS, 1973), and a procedure developed by Huff (1967). The SCS curves are shown in Figure III.2-2. For events shorter than 24 hours,

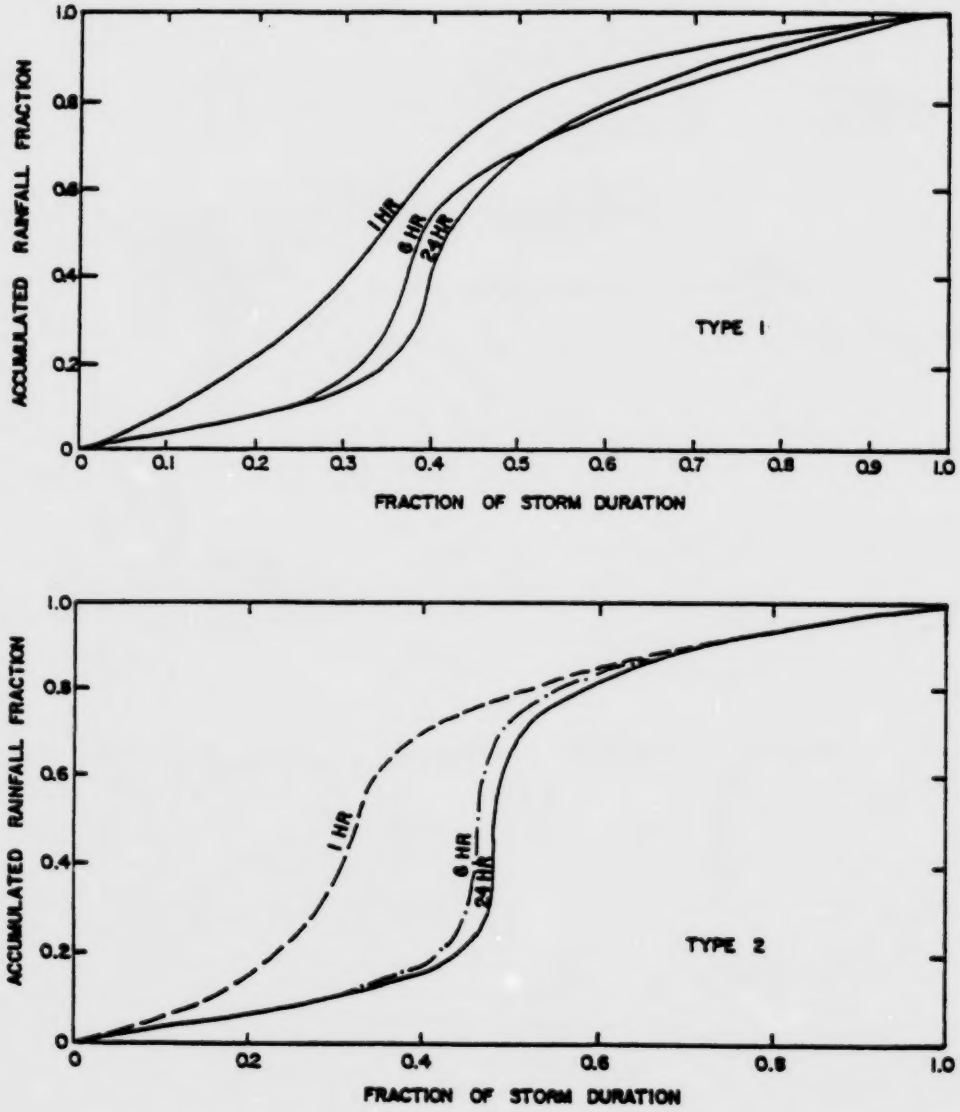


FIGURE III.2-2 SOIL CONSERVATION SERVICE TYPE I AND II RAINFALL DISTRIBUTIONS (ADAPTED FROM WARD ET AL., 1980)

the most intense portion of a curve, corresponding to the duration of interest, is used. Huff's set of curves are presented in Figure III.2-3.

Numerous methods have been developed for calculating rainfall excess (volume of runoff). Infiltration methods are discussed in Chapter III.6.2. However, only a few computer models incorporate infiltration methods as a means for determining rainfall excess. The most commonly used method is the SCS curve number procedure (SCS, 1972). In this approach infiltration losses are combined with surface storage by the relationship:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (2.7)$$

where Q is the accumulated runoff or rainfall excess in mm, P is the rainfall depth in mm, and S is a parameter given by

$$S = 25.4\ (1000/CN - 10) \qquad (2.8)$$

where CN is known as the curve number. Typical curve numbers are shown in Table III.2-2 and the four hydrologic soil groups are defined in Table III.2-3.

Time-Area Method

The time-area method uses a conceptual model of the runoff process. The approach is summarized in Figure III.2-4. The time-area diagram is obtained by drawing isochrones on the watershed with the isochrones separated by time t (see Figure II.2-4(a)). The value of time on each isochrone represents the travel time of water from the isochrone to the watershed outlet. The runoff process is envisioned as a translation of the rainfall from the point it strikes the watershed to the watershed outlet with a travel time defined by the time-area diagram. Incremental runoff hydrographs can then be developed for each increment of rainfall excess (ie1, ie2 etc.). A procedure such as the SCS curve number procedure is used to determine rainfall excess. The storm hydrograph is then obtained by combining all the incremental hydrographs.

The Kinematic Approach

This physically based method is theoretically the most complete. The rainfall excess hyetograph is routed as overland flow to established channels and as channel flow to the basin outlet. The method is based on solution of the continuity and momentum equations, as well as, relationships between the geometric characteristics (such as slope, roughness, channel shape, etc) of the watershed and drainage systems. Details of the method can be found in any recent hydrology text.

Unfortunately, the method is data intensive and difficult to use. The approach has mainly seen application in urban areas where the watersheds are small and highly channelized.

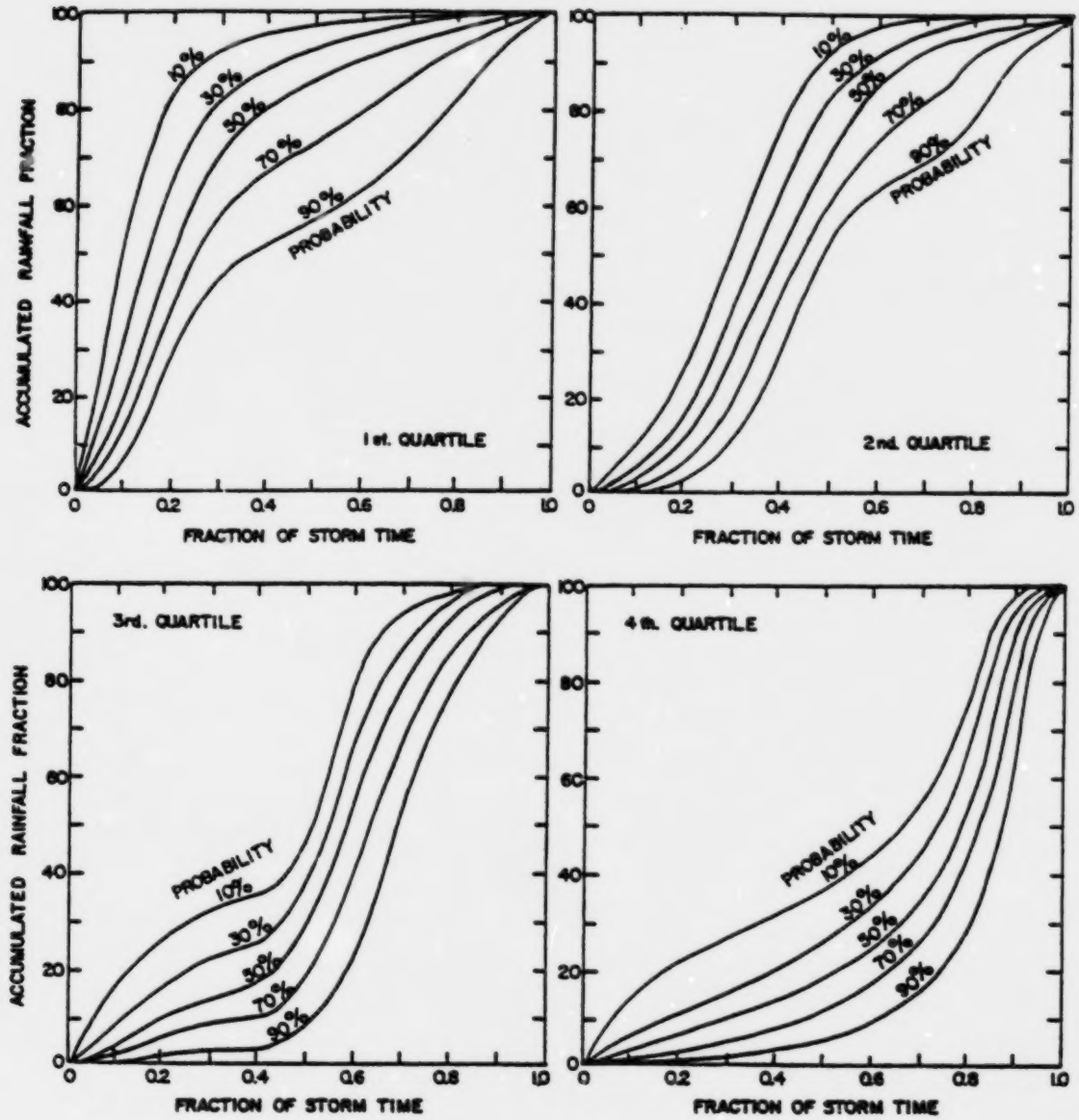


FIGURE III.2-3 HUFF'S STORM DISTRIBUTIONS (ADAPTED FROM HUFF, 1967)

TABLE III.2-2 RUNOFF CURVE NUMBERS FOR HYDROLOGIC SOIL-COVER COMPLEXES
(Antecedent moisture condition II, and I_a=0.2 S)

LAND USE AND TREATMENT OR PRACTICE	HYDROLOGIC CONDITION	HYDROLOGIC SOIL GROUP A	B	C	D
Fallow					
Straight row	----	77	86	91	94
Row crops					
Straight row	Poor	72	81	88	91
Straight row	Good	67	78	85	89
Contoured	Poor	70	79	84	88
Contoured	Good	65	75	82	86
Contoured and terraced	Poor	66	74	80	82
Contoured and terraced	Good	62	71	78	81
Small grain					
Straight row	Poor	65	76	84	88
Straight row	Good	63	75	83	87
Contoured	Poor	63	74	82	85
Contoured	Good	61	73	81	84
Contoured and terraced	Poor	61	72	79	82
Contoured and terraced	Good	59	70	78	81
Close-seeded legumes or rotation meadow					
Straight row	Poor	66	77	85	89
Straight row	Good	58	72	81	85
Contoured	Poor	64	75	83	85
Contoured	Good	55	69	78	83
Contoured and terraced	Poor	63	73	80	83
Contoured and terraced	Good	51	67	76	80
Pasture or range					
No mechanical treatment	Poor	68	79	86	89
No mechanical treatment	Fair	49	69	79	84
No mechanical treatment	Good	39	61	74	80
Contoured	Poor	47	67	81	88
Contoured	Fair	25	59	75	83
Contoured	Good	6	35	70	79
Meadow	Good	30	58	71	78
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	25	55	70	77
Farmsteads	----	59	74	82	86
Roads[1]					
Gravel	----	76	85	89	91
Dirt	----	72	82	87	89
Hard surface	----	74	84	90	92
Industrial Districts (72% impervious)	----	81	88	91	93

[1] Including rights-of-way.

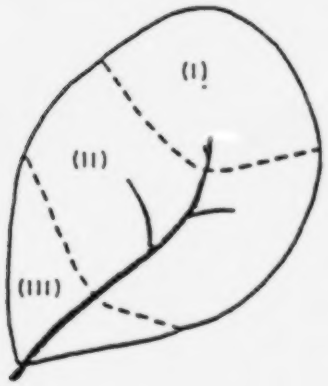
TABLE III.2-3 Definition of SCS Hydrologic Soil Groups

A. These soils have a high infiltration rate. They are chiefly deep, well-drained sands or gravels. (Low runoff potential).

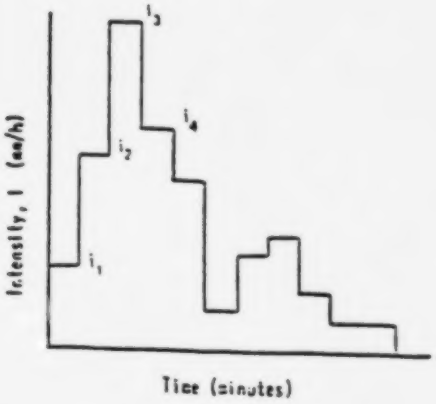
B. These soils have a moderate infiltration rate when thoroughly wet. They are chiefly moderately deep, well-drained soils of moderately fine to moderately course texture.

C. These soils have a slow infiltration rate when wet. They are chiefly moderately deep, well-drained soils of moderately fine to moderately course texture.

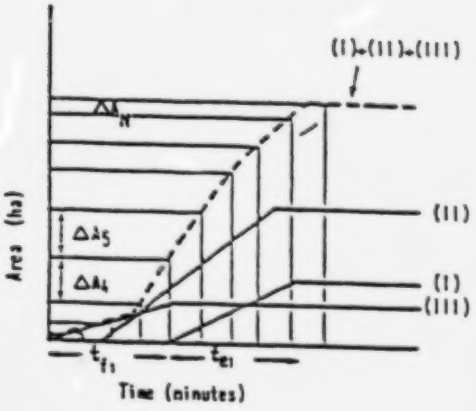
D. These soils have a very slow infiltration rate. They are chiefly clay soils with a high swelling potential, soils with a permanently high water table, soils with a clay pan at or near the surface and shallow soils over nearly impervious materials. (High run-off potential).



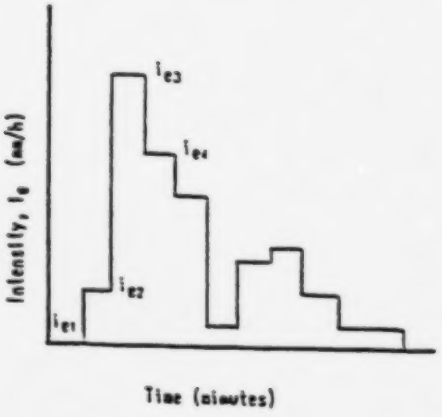
(a) Catchment



(b) Rainfall



(c) Time-area diagram



(d) Excess rainfall

$$Q_1 = \frac{1}{360}(i_{e1} \cdot \Delta A_1)$$

$$Q_2 = \frac{1}{360}(i_{e2} \cdot \Delta A_1 + i_{e1} \cdot \Delta A_2)$$

$$Q_3 = \frac{1}{360}(i_{e3} \cdot \Delta A_1 + i_{e2} \cdot \Delta A_2 + i_{e1} \cdot \Delta A_3)$$

$$Q_n = \frac{1}{360}(i_{en} \cdot \Delta A_1 + i_{en-1} \cdot \Delta A_2 + \cdots + i_{en-2} \cdot \Delta A_N)$$

(e) Computation of hydrograph

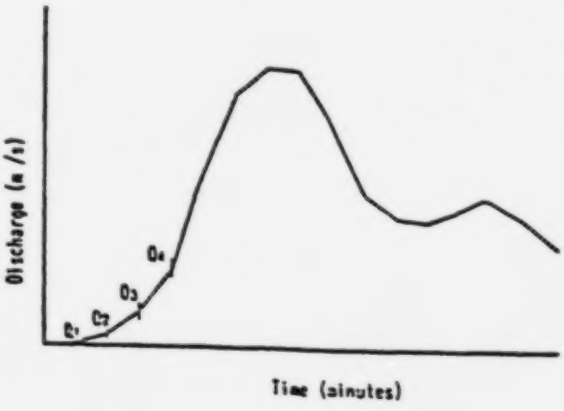


FIGURE III.2-4 TIME - AREA METHOD

III.2.2 FLUVIAL GEOMORPHIC INFLUENCES

Schumm (1977) considers an ideal fluvial system as consisting of 3 zones. These are indicated in Figure III.2-5. Zone 1 is the production zone which forms the drainage watershed which supplies most of the streamflow. Zone 2 is a transfer zone wherein the water collected in Zone 1 is transferred to the depositional Zone, Zone 3.

With the emphasis on minimizing the upstream drainage area above uranium tailings impoundments, most sites developed during the last decade have been located in Zone 1. A few inactive sites are located in Zone 2.

In Zone 1 the major factor of concern is erosion and the potential for gully encroachment on the impoundment. These aspects are discussed in Section III.2.4. In Zone 2, flood intrusion and the potential for river channel changes to intrude on the impoundment are of concern. This topic is covered in detail by Schumm in "Geomorphic Controls on the Management of Nuclear Waste" (Schumm and Chorley, 1983). A brief overview follows.

Identification of Fluvial Instability

Factors influencing river morphology include bed-load, suspended load, sediment particle distribution, channel geometry, flow rates and other geomorphological controls on the river system.

Uniform flow in an open channel with rigid boundaries can be described by a single equation such as Manning's equation (Chow, 1959). For any given discharge, one flow depth will establish itself and the channel is said to have one degree of freedom. However, in a natural channel with erodible sides a set of 3 independent equations is required to define the flow. The channel is said to have 3 degrees of freedom and the flow depths will vary with changes in slope and width associated with erosion of the channel banks. According to Blench (1961) a fourth degree of freedom develops if the river is left to itself. Straight sections will be eroded and the river will naturally meander.

Like all physical systems a drainage system with erodible banks will attempt to be in equilibrium. This process is known as the regime theory and is described by a set of 3 empirical equations. Primarily because of the ability of the system to develop meanders, no exact analytical method exists to define the geomorphic behaviour of a river system.

The original set of equations according to Blench (1961) may be expressed as:

$$F_b = \frac{u^2}{D} \qquad (2.9)$$

where F_b is the bed sediment factor F_b, u is the mean flow velocity and D is the flow depth. This equation is known as the bed factor equation.

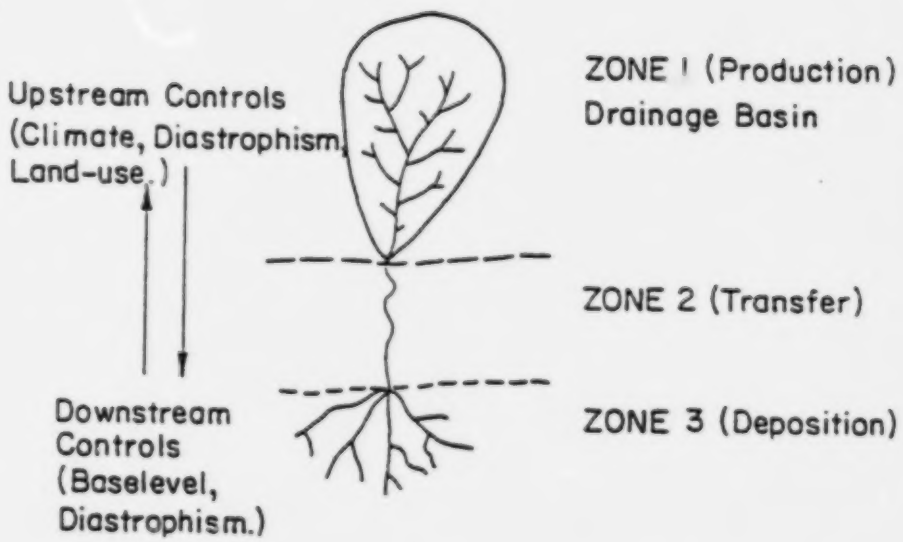


FIGURE III.2-5 IDEALIZED FLUVIAL SYSTEMS

The second equation is:

$$F_s = \frac{v^3}{\frac{A}{D}} \qquad (2.10)$$

where F_s is the side factor and A is the cross-sectional area.

The third equation is:

$$\frac{v^2}{gDS} = 3.63\,(1+aC_s)\left(\frac{vA}{DU}\right)^{0.25} \qquad (2.11)$$

which is known as the regime slope equation, where: S is the bedslope, C_s is the bedload, a is 1/233, and v is the viscosity of fluid-solid mixture.

The regime equations in the most general form for width, depth and velocity are

$$B = C_B Q^a \qquad (2.12)$$

$$D = C_D Q^b \qquad (2.13)$$

$$u = C_U Q^c \qquad (2.14)$$

A summary of regime relations (source Graf,1971) is shown in Table III.2-4.

Leopold and Wolman (1957) noted three different channel patterns, namely, meandering, braided and sinuous. If the ratio between the channel length and the valley length is greater than 1.5, the river is said to meander. A braided river does not have a single or well defined channel and consists of a network of interconnected streams. Braiding is considered an incipient form of meandering. Braided river reaches are generally steeper, wider, and shallower than undivided reaches carrying the same discharge. For any given discharge, meanders occur at smaller slopes.

In assessing the potential hazard a river presents to a tailings impoundment the main item of concern is the potential for the channel to encroach on the impoundment. According to Nelson et al. (1983), types of hazards presented could be:

a) Erosion hazards:
   1) Degradation and Scour
   2) Nickpoint migration

TABLE III.2-4 SUMMARY OF REGIME RELATIONS FOR HYDRAULIC GEOMETRY OF RIVERS

Reporter	*B*	*D*	*ū*	*Location*	*Remarks*
Leopold et al. (1953)	$Q^{0.26}$	$Q^{0.40}$	$Q^{0.34}$	20 river cross sections representing a large variety of rivers in the Great Plains and the Southwest of the U.S. (semiarid conditions)	Variations of hydraulic characteristics in a particular cross section
Leopold et al. (1964)	$Q^{0.01}$	$Q^{0.41}$	$Q^{0.58}$	Brandywine Creek, Pennsylvania (humid eastern U.S.)	Variation of hydraulic characteristics in a particular cross section
Leopold et al. (1964)	$Q^{0.13}$	$Q^{0.46}$	$Q^{0.43}$	Average of 158 gaging stations in the U.S.	Variation of hydraulic characteristics in a particular cross section
Leopold et al. (1953)	$Q^{0.5}$	$Q^{0.4}$	$Q^{0.1}$	20 river cross sections representing a large variety of rivers in the Great Plains and the Southwest of the U.S. (semiarid conditions)	Variation of hydraulic characteristics in a downstream direction; for mean annual discharge
Leopold et al. (1964)	$Q^{0.19}$	$Q^{0.41}$	$Q^{0.40}$	10 gaging stations on the Rhine River	Average at a station relation
Leopold et al. (1956)	$Q^{0.5}$	$Q^{0.3}$	$Q^{0.3}$	Ephemeral streams in semiarid region of New Mexico, U.S.	Variation of hydraulic characteristics in a downstream direction; for mean annual discharge
Nash (1959)	$Q^{0.50}$	$Q^{0.37}$	$Q^{0.33}$	British and U.S. data from Nixon (1959)	For bankfull discharge
Rybkin (1947)	$Q^{0.57}$	$Q^{0.35}$	$Q^{0.31}$	Rivers in the Upper Volga and Oka Basin	Mean long-term discharge
Langbein (1964)	$Q^{0.43}$	$Q^{0.37}$	$Q^{0.10}$	—	Theoretically
Blench (1957)	$Q^{0.50}$	$Q^{0.35}$	$Q^{0.17}$	—	Regime equations for canals

b) Deposition hazards:
   3) Aggradation and fill
   4) Downfilling and backfilling

c) Pattern-change hazards
   5) Meander growth and shift
   6) Channel bars and islands
   7) Cutoffs
   8) Avulsion

d) River metamorphosis hazards
   9) Straight to meandering
   10) Straight to braided
   11) Braided to meandering
   12) Braided to straight
   13) Meandering to straight
   14) Meandering to braided

Horizontal and vertical instability might occur in a river. Horizontal stability will result in the river type changing. As discussed earlier the type of channel which will be formed is a function of the sediment load, bed slope, sinuosity and flow rate. Relationships between these factors are presented in Figures III.2-6, III.2-7 and III.2-8. These Figures can be used to assess the horizontal stability of a channel.

Vertical instability will result in a downcutting of the channel. Downcutting can cause a change in river type or erosion. Generally, downcutting is only significant in alluvial channels. The bed and banks of alluvial channels are formed from deposited material transported by the channel. Table III.2-5 provides a classification of alluvial channels and indicates their relative stability.

Flood Intrusion Impacts and Protection Measures

The impact of flood intrusion will depend on the proximity of the river to the impoundment, flow velocities and the protection characteristics of the toe of the impoundment. If flood flows come in contact with the toe, some degradation should be expected. This should be considered unacceptable unless protective measures (such as the placement of a riprap cover) have been installed at the toe.

It should be noted that adequate protective measures cannot be taken unless the behaviour of the river can be predicted. Therefore it must first be established if vertical or horizontal instability in the river is a factor. The possible impact of changes in the river on the impoundment must be assessed. If they could be a factor, control measures need to be constructed along the river course or protective measures taken at the toe of the impoundment.

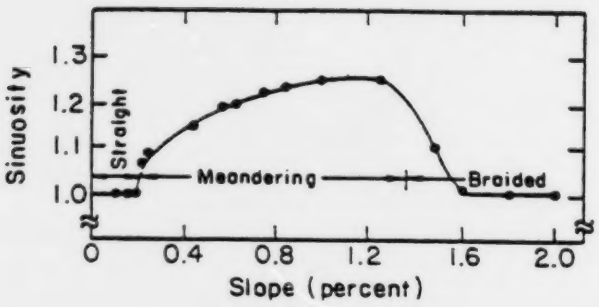


FIGURE III.2-6 RELATION BETWEEN CHANNEL SINUOSITY AND FLUME SLOPE (FROM SCHUMM AND KHAN, 1972)

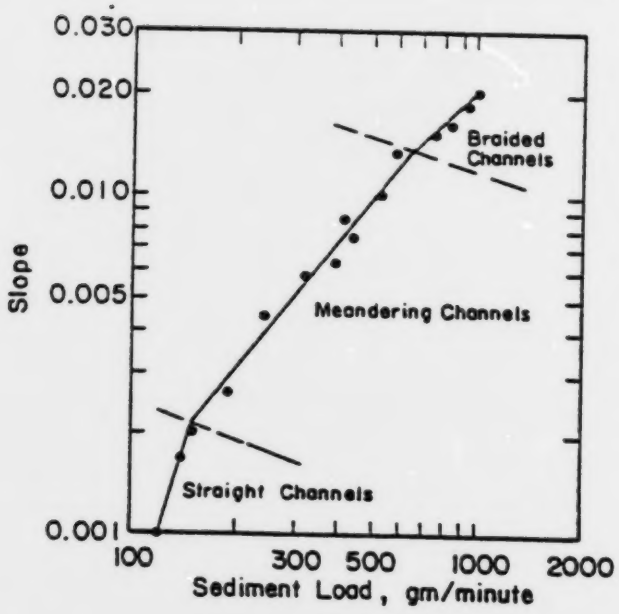


FIGURE III.2-7 RELATION BETWEEN SEDIMENT LOAD AND FLUME SLOPE SHOWING INCREASED RATE OF SEDIMENT TRANSPORT AT THRESHOLDS OR PATTERN CHANGE (FROM SCHUMM AND KHAN, 1972)

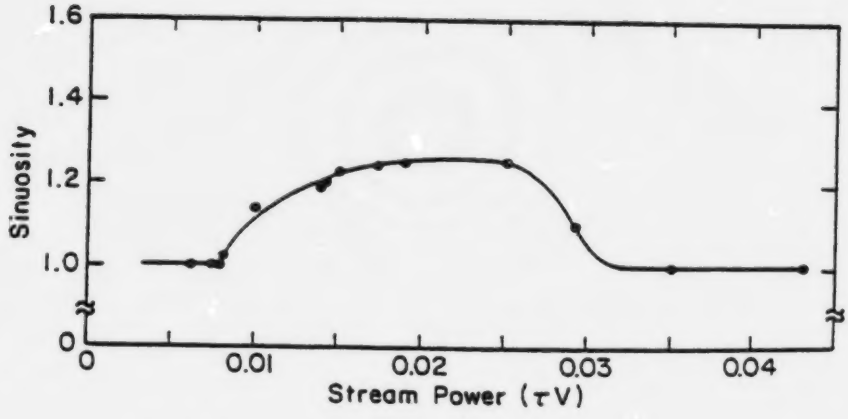


FIGURE III.2-8 RELATION BETWEEN SINUOSITY AND STREAM POWER (DATA FROM KHAN, 1971)

TABLE III.2-5 CLASSIFICATION OF ALLUVIAL CHANNELS

Mode of sediment transport and type of channel	Channel sediment (M) (percent)	Bedload (percentage of total load)	Channel stability		
			Stable (graded stream)	Depositing (excess load)	Eroding (deficiency of load)
Suspended load	>20	<3	Stable suspended load channel. Width/depth ratio <10; sinuosity usually >2.0; gradient relatively gentle	Depositing suspended load channel. Major deposition on banks cause narrowing of channel; initial streambed deposition minor	Eroding suspended-load channel. Streambed erosion predominant initial channel widening minor
Mixed load	5-20	3-11	Stable mixed-load channel. Width/depth ratio >10, <40; sinuosity usually <2.0; >1.3; gradient moderate	Depositing mixed-load channel. Initial major deposition on banks followed by streambed deposition	Eroding mixed-load channel. Initial streambed erosion followed by channel widening
Bed load	<5	>11	Stable bed-load channel. Width/depth ratio .>40; sinuosity usually <1.3; gradient, relatively steep	Depositing bed-load channel. Streambed deposition and island formation	Eroding bed-load channel. Little streambed erosion; channel widening predominant

III.2.3 SOIL EROSION

Erosion and sedimentation by water embody the processes of detachment, transportation, and deposition of soil particles by the erosive and transport agents of raindrop impact and runoff over the soil surface (ASCE, 1975). Erosion can occur as:

1) Sheet or interrill erosion: which is the removal of a thin relatively uniform layer of soil particles

2) Rill erosion: which is erosion in numerous small channels that are a few millimeters wide and deep

3) Gully erosion: gullies are rills which cannot be obliterated by normal tillage practices

4) Stream channel erosion: erosion occurs due to the streamflow

5) Landslide erosion: erosion occurs as a mass soil movement due to slope instability.

For the purpose of this discussion sheet and rill erosion will be discussed as a single topic as will gully and stream channel erosion. Gully erosion is discussed in Section III.2.4 and slope stability is discussed in Chapter III.7.0.

Detachment of soil particles is basically a function of the erosive forces of rainfall impact and flowing water, the susceptability of the soil to detachment, the presence of material that reduces the magnitude of the eroding forces, and management of the soil that makes it less susceptible to erosion (Foster and Meyer, 1972). Transport is a function of the transportability of the detached particles, transport forces, and the presence of materials that reduce transport forces.

Either detachment or transport capacity may limit erosion and sediment load at a location on a slope, as shown on Figure III.2-9. At a given location the amount of sediment transported will either be the amount available or the transport capacity, whichever is less. Sediment movement therefore, can be divided into an erosion component and a delivery component. Soil particles are detached in the erosion process and made available for transport. A transport capacity must exist to deliver the eroded particles downstream. If the transport capacity is less than the gross amount eroded, deposition will occur.

Techniques used to predict sediment movement can be divided into two groups (University of Kentucky, 1983). In the first group the erosion and delivery components are considered separately and then combined to estimate the sediment movement. In the second group both the detachment and delivery of soil particles is considered in a single model.

Several physically based models of the erosion process have been developed (Gregory et al., 1977; Meyer and Wischmeier, 1969; Foster and Meyer, 1972; Crosby and Overton, 1979; and McWhorter et al., 1979). However, these models have only found limited application as predictive methods primarily

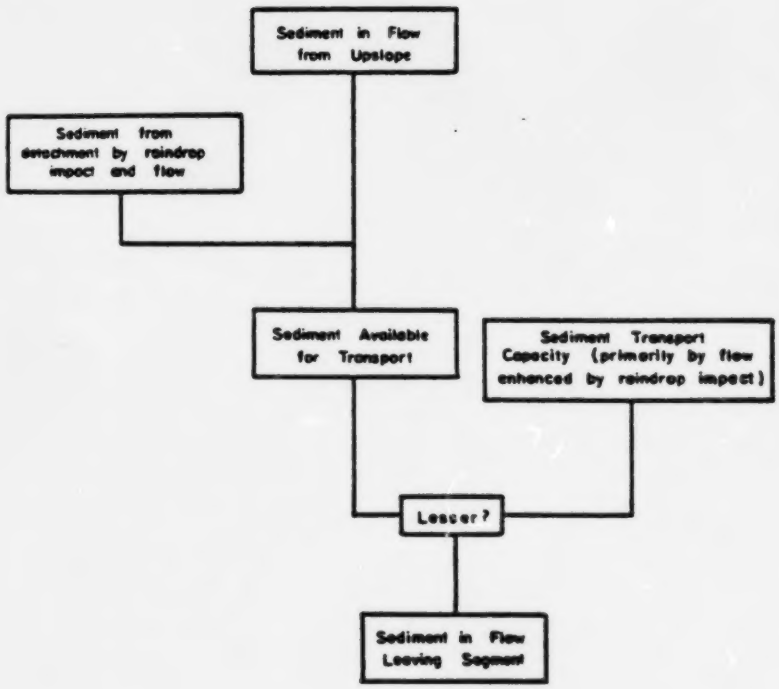

Sediment in Flow from Upslope
Sediment from detachment by raindrop impact and flow
Sediment Available for Transport
Sediment Transport Capacity (primarily by flow enhanced by raindrop impact)
Lesser?
Sediment in Flow Leaving Segment

FIGURE III.2-9 INTERACTION OF EROSION PROCESSES AT STEADY STATE ON A TYPICAL LAND PROFILE (MEYER AND WISCHMEIER, 1969)

because they are data intensive and not suitable for application in large uncalibrated watersheds. A good account of these models is given by Wilson in the SEDIMOT II Design Manual (University of Kentucky, 1983).

Universal Soil Loss Equation

The most commonly used method for predicting erosion is the Universal Soil Loss Equation (USLE) (Wischmeier and Smith, 1965). The method is described by the equation:

$$A = R\ K\ LS\ C\ P \qquad (2.15)$$

where: A is the soil loss per unit area, R is the rainfall factor usually expressed as the product of rainfall energy times the maximum 30 minute intensity for a given rainstorm, K is the soil erodibility, LS is a dimensionless slope length factor, C is a dimensionless soil cover factor and P is a dimensionless conservation practice factor. A very detailed account of the determination of these factors is presented by Barfield et al. (1981). Parameters can be estimated from the information provided in Appendix III.2.1.

The USLE should only be used to determine gross erosion from an area. To determine the sediment yield at a downstream point the gross erosion needs to be multiplied by a sediment delivery ratio (sediment yield/gross erosion). Numerous methodologies have been developed to predict sediment delivery ratios. Regional regression equations have been developed for a number of American watersheds (ASCE, 1977). However, these techniques have only a limited application and should not be applied for Canadian conditions unless (or until) appropriate regional regression equations have been developed.

Barfield et al. (1981) have suggested the following conceptual model for surface mined watersheds:

$$DR = DA * DV * DC * DP \qquad (2.15a)$$

where: DR is the delivery ratio and DA, DV, DC and DP are area, vegetative, channelization, and deposition effects, respectively, on the delivery ratio. Furthermore they suggest that the selection of delivery ratio should be dependent on the method used to estimate upland soil erosion. For example, the Colorado State Model (CSU) should use a different delivery ratio than the USLE because the CSU model already accounts for deposition of sediment in upland areas. While this method has a lot of merit there is only a limited amount of information on the selection of the DA, DV, DC and DP factors.

Modified Universal Soil Loss Equation

Williams (1975, 1977 and 1978) has developed empirical procedures which allow regional equations to be developed without gathering costly field data. Procedures based on these approaches have been incorporated in the SEDIMOT II model (University of Kentucky, 1983) and the WASHMO model (Ward et al., 1981) and are discussed further in the following section.

A method known as the Modified Universal Soil Loss Equation (MUSLE) (Williams, 1975) overcomes the problem of determining a delivery ratio and has seen widespread application. The method is defined by the equation:

$$Y = 11.8\ (Q\ q)^{0.56}\ K\ LS\ C\ P \qquad (2.16)$$

where: Y is the single storm sediment yield in tonnes, Q is the storm runoff volume in cubic metres, q is the peak storm discharge in cubic metres per second and K, LS, C and P are the standard USLE terms.

Williams (1975) proposed a simple mathematical model to route sediment generated in subcatchments through drainage systems to the watershed outlet. Sediment yield for each subwatershed is determined using MUSLE and all sediment particles are routed by a single representative particle size. It is assumed that the change in sediment load is a function of the square root of this particle diameter, the amount of sediment load in the flow, and the time available for deposition. These relationships are represented by the equation:

$$Y = Yo\ e^{-B\cdot T\sqrt{D_{50}}} \qquad (2.17)$$

where: Yo is the sediment yield at an upstream section and T is the travel time between the subwatershed and the watershed outlet. The total sediment yield at the outlet is estimated by summing the contribution from each subwatershed as predicted by Equation 2.17.

Erosion-Sediment Yield Models

Numerous erosion-sediment yield models have been developed. Table III.2-6 summarizes 13 of the more commonly used models (Haan et al., 1982). A discussion on several of these models has been presented. The CREAMS model has begun to see widespread application on agricultural watersheds. The CSU and ANSWERS models have seen limited application on a variety of different catchments but are still generally used as research tools.

SEDIMOT 2

In recent years two models (SEDIMOT 2 and WASHMO) have begun to see widespread application on watersheds disturbed by mining. The most widely used of these models is the SEDIMOT 2 (SEdimentology by Distributed MOdel Treatment) model which was developed by the Agricultural Engineering Department at the University of Kentucky (University of Kentucky, 1983).

The model is a single event distributed parameter model which simulates the runoff, erosion and transport process on a disturbed watershed. The program is capable of predicting a hydrograph and sedimentgraph for individual subwatersheds. These flows are then routed to and through sediment control structures to evaluate the sediment trap efficiency of the controls.

The rainfall-runoff process is modelled using the SCS techniques discussed earlier. The rainfall depth associated with a design storm event is converted into a temporal storm pattern by selection of the SCS Type 1 or Type 2 24-hours rainfall pattern or by inputting a user developed pattern.

TABLE III.2.6 EXAMPLES OF EROSION - SEDIMENT YIELD MODELS

1. Universal Soil Loss Equation (USLE) (Wischmeier and Smith, 1978):
   - Type: Simple, empirical equation developed from regression analyses of 10,000 plot years of data from natural runoff plots and plots under artificial rainfall simulators.
   - Estimates: Long term, average annual sheet-rill erosion
   - Strengths: Simple, large data base, parameter values readily available, widely used by agencies like the USDA-Soil Conservation Service, adaptable to nonuniform areas where deposition does not occur.
   - Limitations: Does not estimate deposition, sediment yield, channel erosion, or gully erosion; is not accurate for a single storm event.
2. Onstad and Foster (1975):
   - Type: A modification of the USLE where the rainfall erosibity factor has been replaced with one having separate terms for rainfall and runoff.
   - Estimates: Erosion for single storm event on either a uniform slope or a watershed.
   - Strengths: Simple, easy to apply, uses the USLE data base and parameter data set.
   - Limitations: No information on distribution of erosion during event; does not estimate deposition, sediment yield, or any form of channel erosion.
3. Modified Universal Soil Loss Equation (MUSLE) (Williams, 1975a):
   - Type: A modification of the USLE where the R factor has been replaced with a runoff erosivity factor.
   - Estimates: Sediment yield from watersheds for an individual storm event.
   - Strengths: Simple, easy to apply, uses the USLE parameter data set.
   - Limitations: Provides no information on time distribution of sediment yield during a runoff event, is strictly a sediment yield equation and should not be used where detachment controls sediment yield.
4. Negev (1967):
   - Type: Based on relationships for erosion subprocesses and driven by Stanford Watershed Model (See David and Beer and ARM models for additional discussion of this type). This was an early application of using a detailed hydrology model to drive an erosion model.
5. Meyer and Wischmeier (1969):
   - Type: Has relationships for each of the major erosion subprocesses occurring on overland flow areas. Was an early model which formularized fundamental erosion concepts. Curtis (1976) described application of this model.
6. Lane and Shirley (1978):
   - Type: Analytically derived model based on fundamental erosion mechanics and kinematic overland flow assumptions.
   - Estimates: Erosion from uniform slopes for single storms.
   - Strengths: Potential for development into a practical erosion equation based on fundamental erosion processes, relatively simple, can be adapted to nonuniform slope shapes.
   - Limitations: Restricting hydrologic assumption like uniform rainfall excess and simple overland flow slopes, parameter data set not available.
7. David and Beer (1975), ARM (Donigian and Crawford, 1976; Donigian and Davis, 1978; Fleming and Leythan, 1976):
   - Type: Has relationships for major erosion subprocesses for both overland flow and channel areas except ARM which does not consider channels, driven by Stanford Watershed Model or one of its variations, continuous simulation.
   - Estimates: Erosion and sediment yield for watershed.
   - Strengths: Considers the erosion-sediment yield subprocesses and uses hydrologic inputs for both rainfall and runoff to get interaction effects.
   - Limitations: Lumps parameter values over the watershed, requires historical data to calibrate, questionable transferability to ungaged areas and to land uses significantly different those used during model calibration.
8. Curtis (1976):
   - Type: Uses Meyer and Wischmeier's (1969) erosion relationships and is driven by a kinematic hydrology model using a system of overland flow and channel segments.
   - Estimates: Erosion and sediment yield distribution in time and space for a given event.
   - Strengths: Has relationships to reflect interactions between detachment and transport by rainfall and runoff, is a distributed model able to reflect interactions due to spatial variability.
   - Limitations: Oriented primarily to urban watersheds (Meyer and Wischmeier's relationships have more general application), requires calibration, includes no channel erosion processes.

TABLE III.2.6 (continued)

9. Smith (1977):
   - Type: Based on fundamental erosion and sediment transport relationships and is driven by a kinematic runoff model. Uses a system of a converging overland flow section, overland flow planes, and a channel. An event based model.
   - Estimates: Erosion and sediment yield distributed in time and space.
   - Strengths: Is adaptable to small watersheds like farm fields; is a distributed model; and is fundamentally based.
   - Limitations: Must be calibrated, relationships need to be developed for use as a practical model.
10. Bruce et al. (1975):
   - Type: Is a two stage convolution model similar to instantaneous unit hydrograph theory but model operates nonlinearly, driven by a two stage convolution scheme. An event based model.
   - Estimates: Erosion and sediment yield distributed in time and space.
   - Strengths: Can fit a variety of areas complex in size, shape, and land use.
   - Limitations: Model is abstract and thus difficult for the user to understand, requires historical data to calibrate, transferability to ungaged areas or to new land uses may be limited at present.
11. CSU—(Li, 1977):
   - Type: Based on equations for separate erosion and transport processes on overland flow and channel areas, driven by a kinematic overland flow model, uses a system of grids and channels. An event based model.
   - Estimates: Erosion and sediment yield distribution in time and space, particle distribution of sediment.
   - Strengths: Adaptable to the variety of erosion processes occurring on a variety of overland flow and channel areas. Complex and large areas can be represented. Grids can be combined to eliminate some redundancy in input.
   - Limitations: Although values are available for most parameters, some calibration is required for a few parameters. Although the model can represent complex areas, there may be distortion of parameters depending on how grid size and channel network are chosen to represent watershed. Also, see limitations for ANSWERS model, for additional discussion of problems due to location of grid lines.
12. ANSWERS—(Beasley, 1977):
   - Type: A distributed model that uses separate equations for detachment and transport on overland flow areas and transport in channels. Uses a regular square grid and channels to represent watershed.
   - Estimates: Erosion and sediment yield distributed in time and space for single events.
   - Strengths: Well adapted to remote sensing and other inventory surveys where observations are on a regular grid to determine parameter values, layout of grid is straight forward, represents a variety of land use, soils, and topographic features over the watershed. Model is designed for little or no calibration by using several of the USLE parameters.
   - Limitations: Model currently allows no erosion in channels, model requires great effort to assemble input data. Grid boundaries may not correspond to boundaries of soil or land use, for example, the edge of a grass buffer strip. Or, grid elements may be larger than a subwatershed feature which has a major effect, thereby requiring parameter distortions to avoid errors in the estimates.
13. CREAMS—(Foster et al., 1980; See Simons et al., 1977 for similar model of this type that does not consider channels or impoundments.):
   - Type: Event based model that uses storm EI, volume of runoff, and peak runoff rate with fundamental erosion relationships to compute detachment and transport on overland flow areas and channels in field sized areas. Uses typical overland flow profile, typical channel, and typical impoundment elements to represent watershed. Model can be used for single events, or for long term simulation.
   - Estimates: Erosion, sediment yield, and distribution of sediment particles from each element.
   - Strengths: Inexpensive simulation of 10 to 30 yr of record. Can describe a broad range of management practices. Does not require calibration since model uses USLE and other readily available parameter data sets. Can represent complex field sized watershed with minimum input detail.
   - Limitations: Can represent variability only in downslope direction, does not provide information during the storm. Flow hydraulics are approximate for gradually varied flow since regression equations are used. Model not as powerful as the complete models like ANSWERS and CSU.

Rainfall patterns ranging in duration from 1-24 hours are determined using the technique described by Ward et al. (1979).

Unit hydrograph procedures are used to convert rainfall excess into a storm hydrograph. Selection between double triangle hydrograph shapes which correspond to forested, agricultural and disturbed land uses is provided. Storm hydrographs are routed to structures and between structures by the Muskingum routing procedure (Koussis, 1978).

Two procedures are incorporated in the model for determining sediment yields and for routing these yields from a watershed. The first procedure is based on the MUSLE method and the routing procedures of Williams discussed earlier. The second procedure is based on detachment and transport capacity concepts. Detachment in interrill and rill areas is estimated by the procedures incorporated in the CREAMS model (Knisel, 1980). The sediment transport capacity of a stream is calculated from Yang's unit stream power equation (Yang and Stall, 1976). The model translates sediment yields into sedimentgraphs by assuming that the sediment concentration is proportional to the runoff volume. The model is capable of evaluating the sediment control performance of check dams, sediment dams and vegetative filters.

The sediment trap efficiency of a check dam depends upon the relationship between the size distribution of the incoming sediment load and the void spaces in the structure. The effectiveness of a check dam is evaluated based on the backwater profile and the particle fall velocity (Hirschi, 1981 and Barfield et al, 1981). The procedure requires knowledge of:

1) Incoming sediment size distribution

2) Design flow characteristics

3) Check dam porosity

4) Channel slope

5) Manning's n

6) Channel geometry

The performance of a sediment pond is determined based on a modification of the DEPOSITS model (Ward et al., 1979). Basic inputs to the model are:

1) Stage-area curve for the impoundment

2) Stage-discharge curve for the spillway system

3) Withdrawal characteristics of the spillway system

4) Particle-size distribution and specific gravity of the sediment load

5) Sediment load or inflow sedimentgraph

6) Degree of dead storage

7) Degree of short-circuiting

The model uses plug flow concepts to determine the sediment trapped in the impoundment. The inflow hydrograph and sedimentgraph is divided into plugs of flow and each plug is routed sequentially through the impoundment. For each plug of flow, a detention time and average depth of flow is determined and sediment concentrations computed for four layers within each plug. Settling between layers is calculated based on Stoke's Law. The model can account for dead storage and short-circuiting and can also be used to simulate deposition associated with a bed load. As an alternative to the DEPOSITS model approach, an impoundment can be considered to operate as a series of continuous stirred reactors.

The performance of vegetative filters are simulated based on algorithms developed by Hayes et al. (1979). As sediment laden flow impinges on a vegetative filter, a reduction in its velocity causes the transport capacity to be lowered, which then results in sediment deposition. Sediment trap efficiency is based on the Reynold's number, particle settling velocities, flow velocity, depth of flow, and the filter length.

Basic inputs are:

1) Filter length, slope and width

2) Vegetation spacing

3) Manning's n

4) Rigidity factor

5) Vegetation height

6) Infiltration rate

Outputs from each of the three sediment control options (check dams, sediment ponds and vegetative filters) are:

1) Outflow hydrograph

2) Outflow sedimentgraph

3) Outflow particle size distribution

4) Control trap efficiency

WASHMO and WASHED Models

These models were developed by WARD et al, 1981, and the techniques contained in the models have been incorporated in the SEDIMOT 2 model. The WASHMO hydrograph model has seen limited application in the United States. Both the WASHSED and WASHMO models have also been applied in South Africa.

In a recent study (SRK, 1985) the WASHMO model was found to be comparable or better than many of the commonly used hydrograph techniques.

The WASHMO storm hydrograph model is based on SCS procedures and a paper presenting the mathematics of the model is contained in Ward et al,1981. The WASHSED model is identical to the WASHMO model except that it contains the following additional features:

1) Sediment load can be determined based on the MUSLE

2) Sedimentgraph is developed by the same approach contained in the DEPOSITS model

3) Development of volume weighted sedimentgraphs

4) Development of sediment load weighted particle size distributions

III.2.4 GULLY EROSION

Gully erosion is defined as the erosion of soil or soft rock material by running water that forms distinct, narrow channels that are larger and deeper than rills and that usually carry water only during or immediately after heavy rains or following the melting of snow or ice (American Geological Institute, 1974). Gully erosion is often rapid and progressive. Walls of the gully are steep and vegetation is usually absent. A gully is formed because the tractive forces exerted by concentrated flowing water exceeds the resistance of the materials on which it is eroding.

Cover materials on tailings impoundments will be vulnerable to gully erosion because they are composed of locally derived materials which have been stockpiled and then recently placed on the impoundment. Natural armouring and vegetation growth normally takes several years to become well established and during this period the risk of gully erosion will be the highest.

Nelson et al. (1983) have suggested that protective measures based on the PMF should prevent gully formation for periods of up to 1000 years. Prediction of gully erosion is made based on tractive force methods or empirical procedures.

For example, empirical relationships which relate drainage area and valley slope can be established as shown in Figure III.2-10 (Patton and Schumm, 1975). Points representing ungullied valley slopes that lie above the line representing the critical slope are considered to be prone to gullying. Unfortunately, this type of approach is not applicable to tailings impoundment covers.

The tractive force method proposed by Begin and Schumm (1979) involves the calculation of the average shear stress at the soil- water interface and comparing the results to an allowable tractive force which is determined by an empirical evaluation of the geomorphology of the area. However, this approach is also not directly applicable to the design or evaluation of tailings impoundment covers.

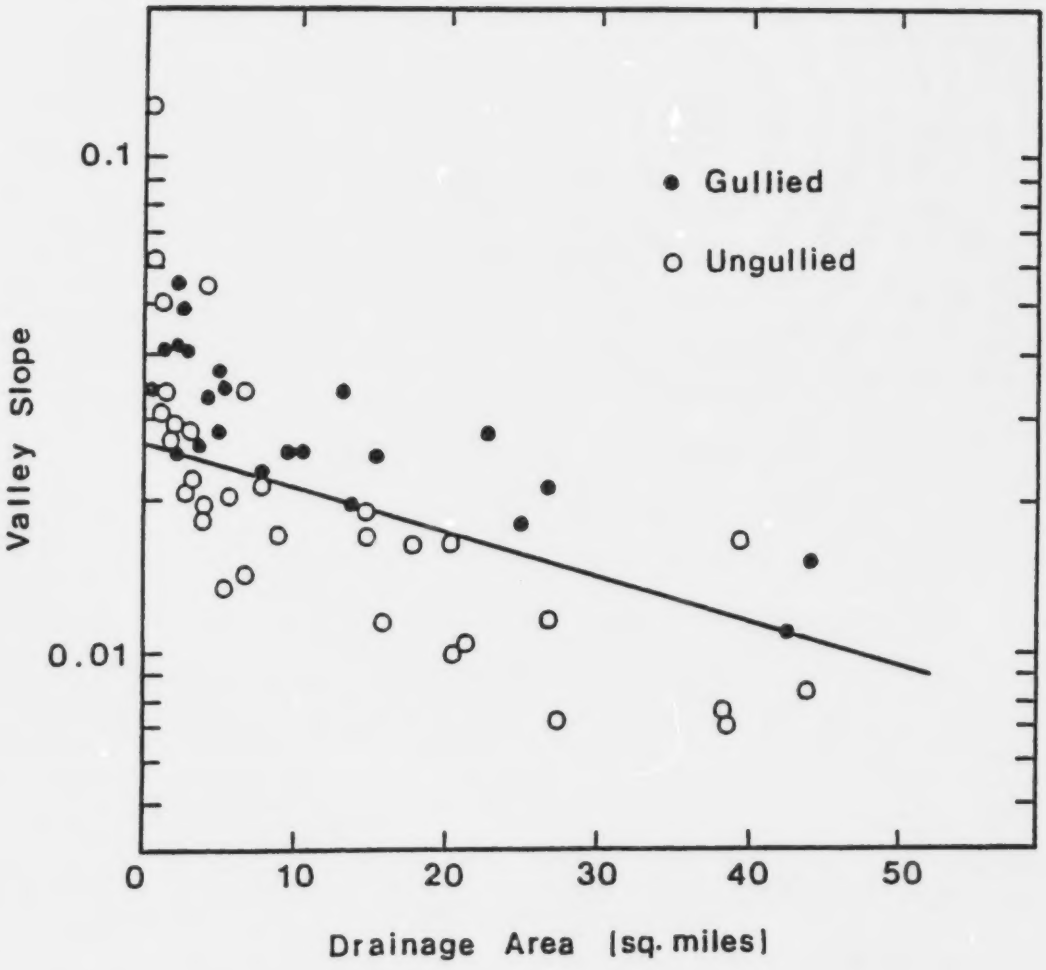


FIGURE III.2-10 CRITICAL VALLEY-FLOOR SLOPE VERSUS DRAINAGE AREA FOR SMALL BASINS, PICEANCE CREEK AREA, WESTERN COLORADO (FROM PATTON AND SCHUMM, 1975)

These type of approaches are best suited to large areas consisting of relatively mature materials. They do not specifically relate the erosion potential to such factors as material particle sizes, cohesion, hardness and durability.

Tractive force methods used for the design of stable open channel systems provide one approach for designing tailings impoundment covers. However, several assumptions have to be made and the effects of vegetation are not adequately accounted for. This approach is illustrated in Section III.2.5.

Gully Intrusion Prediction Procedure

Falk et al. (1985) conducted an extensive field investigation to determine the gully intrusion potential of an impoundment cover. Several reclaimed tailings impoundments which had experienced different levels of gullying were surveyed. Data collection included the measurement of gully and pile dimensions, collection of cover material samples, collection of climatic and precipitation records, and obtaining age and maintenance records.

Based on this data a gully intrusion prediction procedure was developed to estimate the maximum depth of gully incision, the location of the maximum intrusion and the approximate top width of the gully at the point of maximum intrusion. The procedure is based on the following assumptions:

1) the toe of the slope is relatively stable over time

2) the rate of gully erosion and slope degradation decreases with time

3) the material used to reclaim the site is homogeneous

4) differential settlement does not pose a long-term concern

5) the effects of vegetation are considered negligible for vegetative covers of less than 30 percent

A summary of the procedure follows. A more detailed description is presented in Falk et al. (1985). It should be noted that although the approach appears promising it has not yet been verified.

Characteristics of the tailings impoundment side slopes are illustrated in Figure III.2-11. Information requirements include:

1) cover material median particle diameter in mm (D_{50}) and uniformity coefficient, C_u

2) side slope dimensions including the length, L, slope height, H, and the initial gradient, S_i

3) historical rainfall records for the site or nearby vicinity

4) design life of the cover

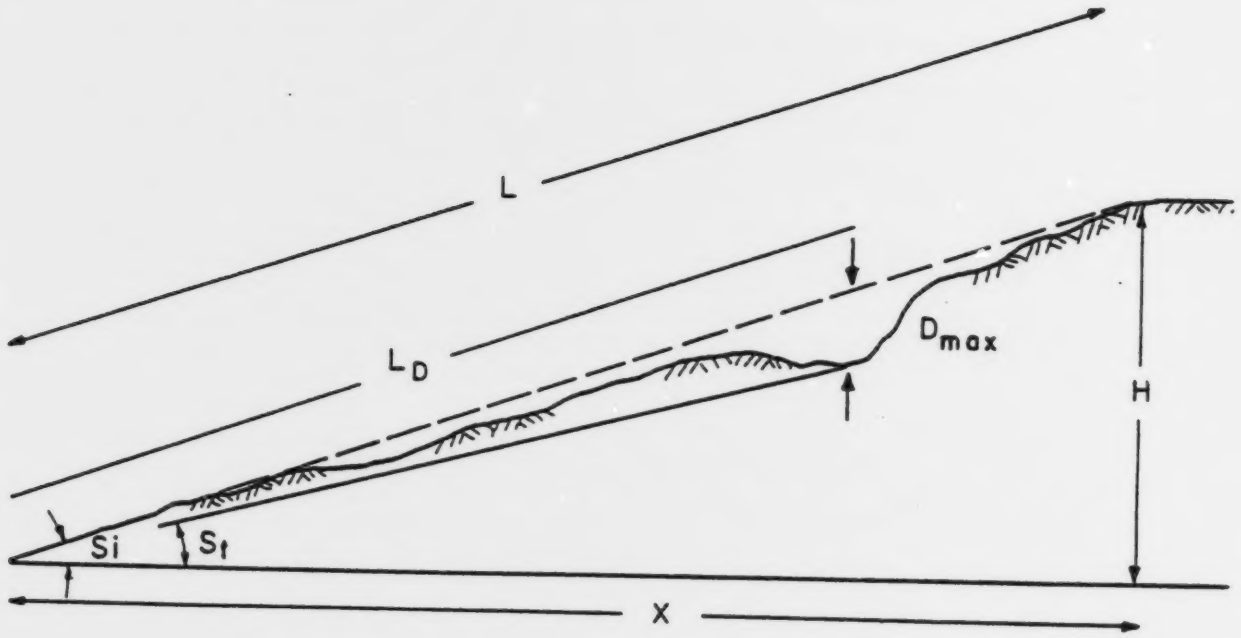


FIGURE III.2-11 CROSS-SECTION OF A TYPICAL RECLAIMED SLOPE ILLUSTRATING GULLY AND ASSOCIATED HEADCUT

It is also necessary to determine the upstream drainage area to any point on the side slope which may serve as a drainage outlet, and to define the longest potential watercourse that traverses the cover to the toe.

The method suggests that the drainage area be determined by using an empirical equation presented by Mosley (1971):

$$D = 0.909 + 22.418\ S_i \qquad (2.18)$$

where D is the drainage density area per unit length of channel (ft^2/ft) and S_i is the design or initial slope of the cover before gullying. The equation is applied to each segment with similar slopes and the total area determined as follows:

$$A_{total} = D_j\ L_j + D_{j-1}\ L_{j-1} + \ldots . \qquad (2.19)$$

where L_j is the potential watercourse length at a slope of S_i for drainage density D_j.

The stable slope S_s is then determined from the equation:

$$S_s = 41.2\ (1 + D_{50}) / (A\ P) \qquad (2.20)$$

where A is the tributary drainage area in square feet, P is the average annual number of precipitation events greater than or equal to 0.5 inches and D_{50} is the median particle size, in mm, of the slope.

During the period of evaluation, the side slope will be between the initial slope, S_i, and the stable slope, S_s. The interim or transitional slope, S_t, is defined as the slope of the tangent extending from the toe to the deepest point in the gully. The transitional slope is calculated as follows:

$$S_t = S_i\ e^{-(GS_s t)} \qquad (2.21)$$

where G is a function of the stable slope and t is the design life of the cover. The coefficient G can be estimated from Figure III.2-12.

The maximum depth of gully incision, D_{max}, is estimated as

$$D_{max} = (LD/L)(H - X\ (S_t + S_s)) \qquad (2.22)$$

where D_{max} is the maximum depth of gully incision, LD/L is the occurrence of D_{max} from the toe and X is the horizontal distance from the toe to the crest of the slope. The occurrence can be determined from Figure III.2-13.

The approximate gully top width can be estimated as:

$$WT = 4.936 + 2.923\ LOG(D_{max}/C_u) \qquad (2.22a)$$

where WT is the top width at D_{max} and C_u is the uniformity coefficient of the cover material.

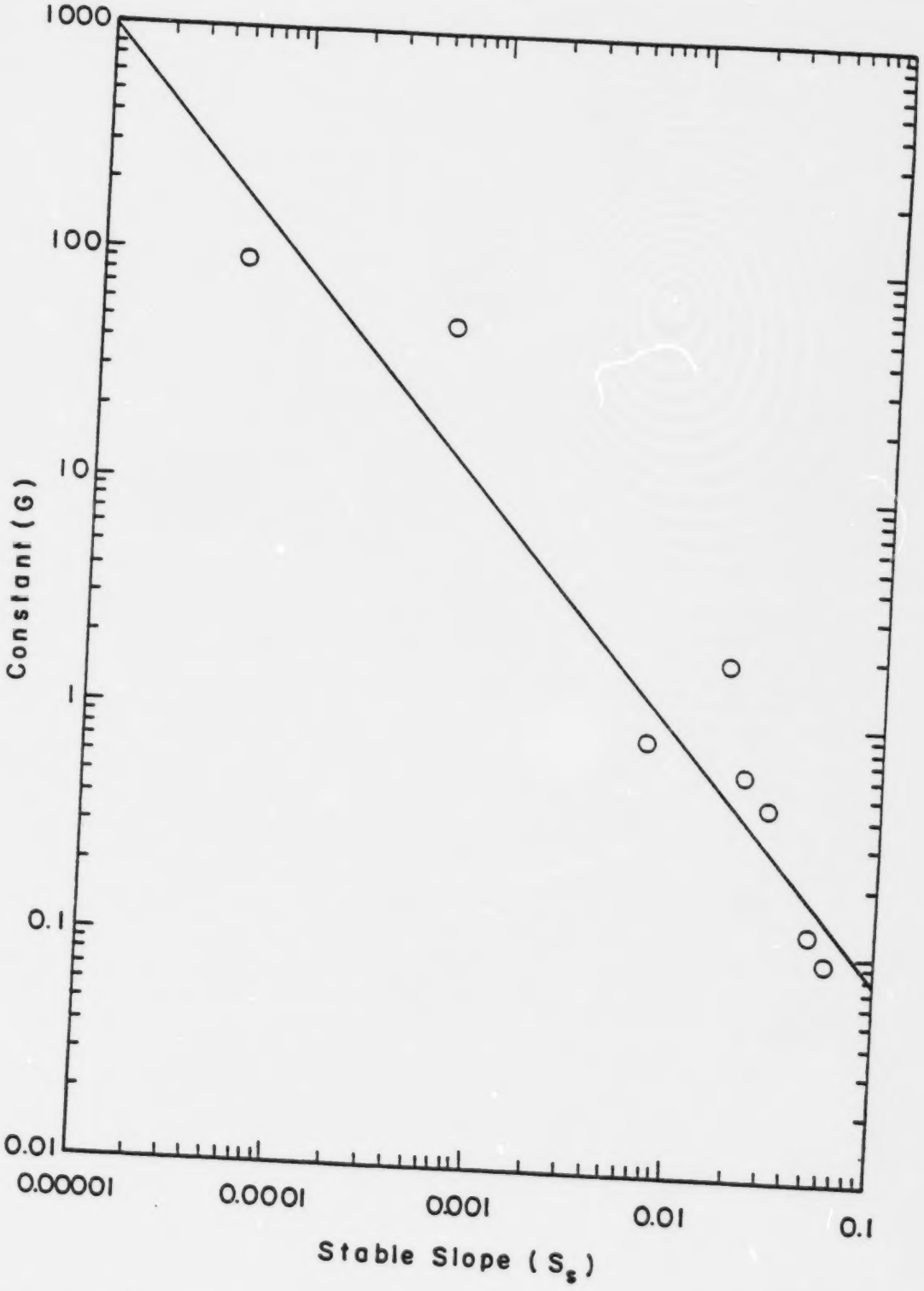


FIGURE III.2-12 RELATION OF STABLE SLOPE TO THE CALIBRATION CONSTANT, G

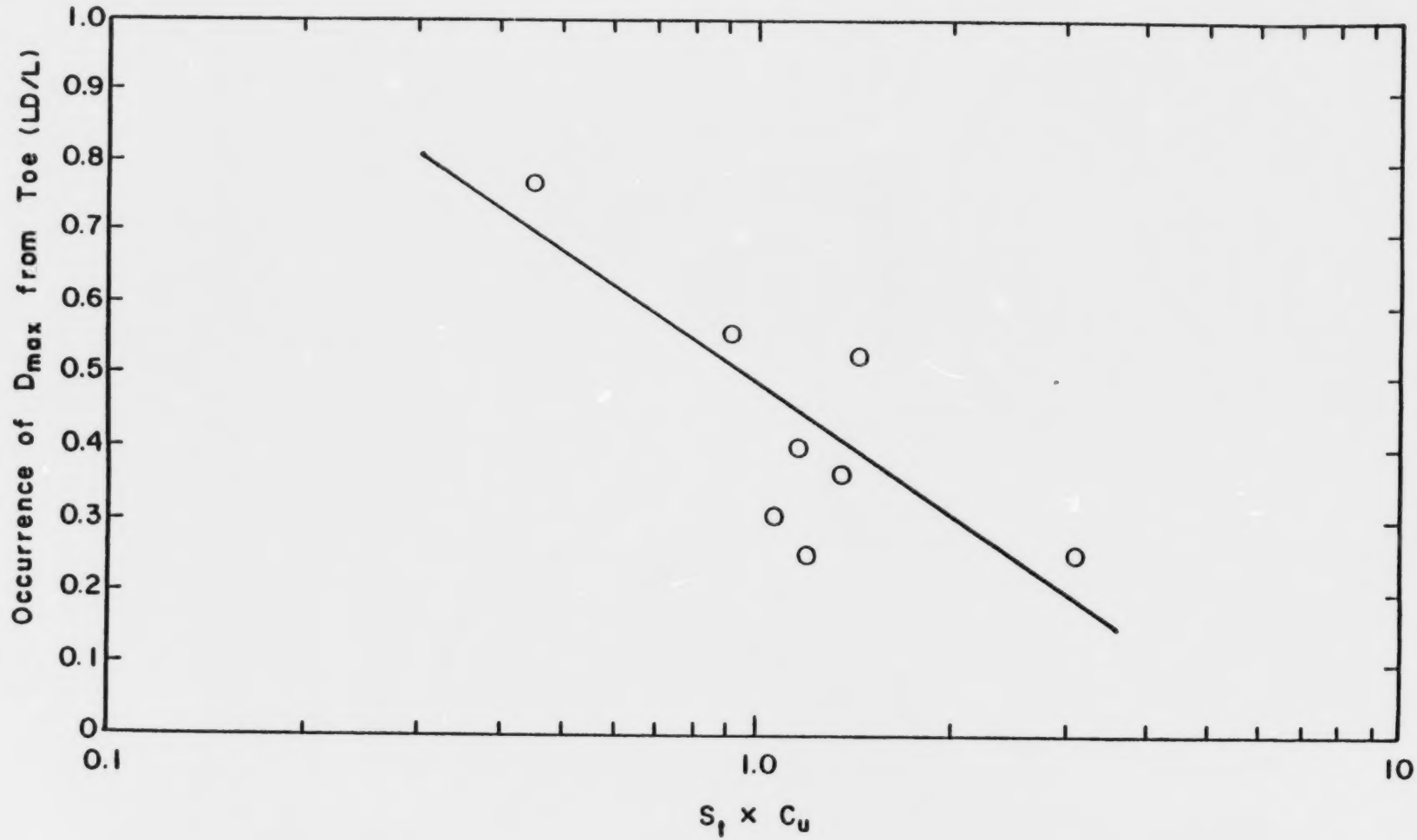


FIGURE III.2-13

RELATION OF TIME - DEPENDENT SLOPE AND UNIFORMITY COEFFICIENT VERSUS OCCURENCE OF MAXIMUM GULLY DEPTH FROM TOE-OF-SLOPE

The developers of the model caution that the approach is very preliminary and should not be used for periods longer than 200 years. An example of the use ofthe approach is presented in Appendix IV.3.1.

III.2.5 DESIGN OF AN EROSION RESISTANT COVER

This approach is purely a design procedure for designing an erosion resistant cover and cannot be used to predict sediment yields or the life of the cover. Two commonly used approaches are the critical shear stress method and the tractive force method.

Critical Shear Stress Approach.

The critical shear stress approach has been developed from open channel slope stability design procedures. The approach consists of providing a cover material with a D_{30} grain size that will resist movement when subject to the sheet flow maximum permissible velocity resulting from the application of the PMP over the entire impoundment surface. Minimum D_{30} grain sizes can be determined by using the procedures presented by Simons and Senturk (1977).

Typically, the minimum construction layer thickness is specified to be at least two times the maximum particle size However, construction constraints will usually require a minimum thickness of 150 to 300 mm (150 to 300 mm). The critical shear stress approach assumes that the cover material will last indefinitely and does not account for weathering.

Tractive Force Approach

The tractive force approach for designing erodible open channels is very similar to the critical shear stress procedure.

For a wide open channel the tractive force can be determined as:

$$r = wyS \qquad (2.23)$$

where r is the unit tractive force, w is the unit weight of water, y is the depth of flow, and S is the slope.

The permissive tractive force is the maximum unit tractive force that will not cause serious erosion of the cover material. Recommended permissible tractive forces for channels are presented in Figures III.2-14 and III.2-15. The adequacy of a cover material or the required particle size of the material, is determined by equating the permissible tractive force to the unit tractive force determined from Equation 2.23.

III.2.6 CONTROL OF EROSION ON TAILINGS IMPOUNDMENT COVERS

A number of methods have been outlined for determining erosion and sediment yields. However, it would be impractical to apply many of them to the design or evaluation of covers for tailings impoundments. Three suitable approaches which have been identified are:

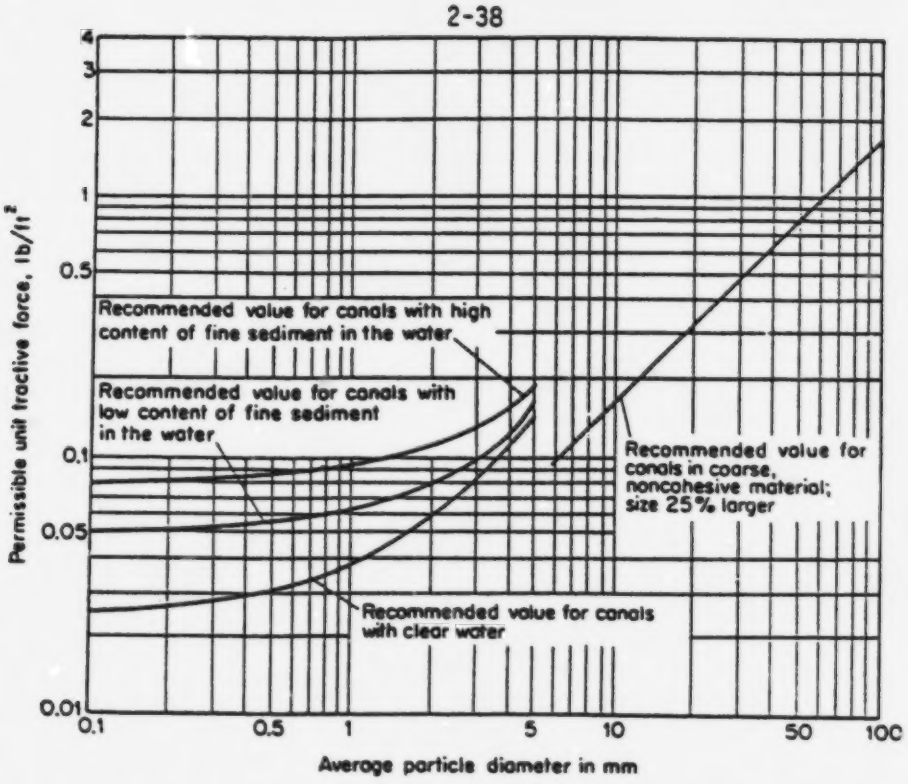


FIGURE III.2-14 RECOMMENDED PERMISSIBLE UNIT TRACTIVE FORCES FOR CANALS IN NONCOHESIVE MATERIAL (U.S. BUREAU OF RECLAMATION)

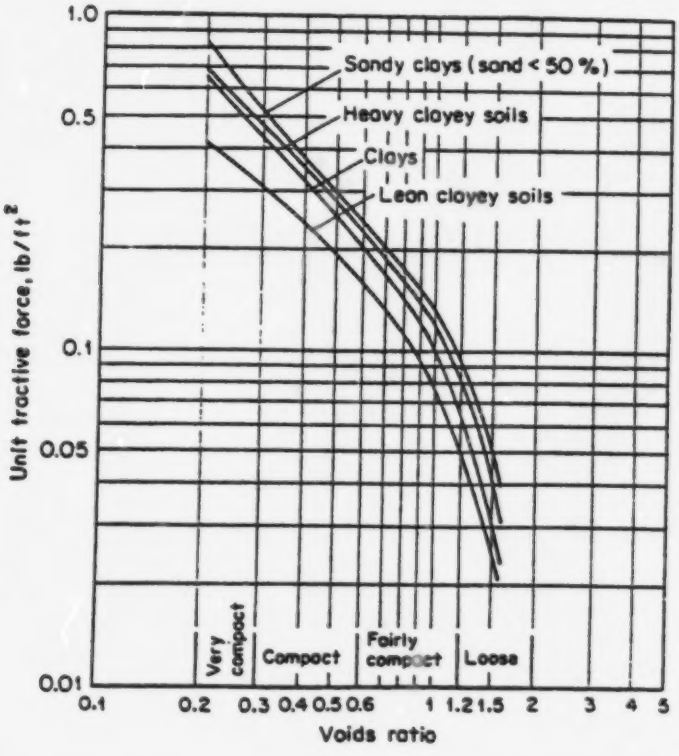


FIGURE III.2-15 PERMISSABLE UNIT TRACTIVE FORCES FOR CANALS IN COHESIVE MATERIAL AS CONVERTED FROM THE U.S.S.R. DATA ON PERMISSABLE VELOCITIES

1) prediction of sheet and rill erosion
   a) USLE
   b) MUSLE

2) prediction of the depth of incision due to gully erosion

3) design of erosion resistant covers

   a) critical shear stress method
   b) tractive force method

The first two basic approaches are discussed briefly below, while the third has been discussed in Section III.2.5.

Rill Erosion Prediction

The USLE can be used to determine average annual erosion losses. These estimates can then be used to determine total loss for the life of the impoundment. If total losses are unacceptable the design can be modified by adjusting such factors as the cover thickness, cover material type, vegetation type, and the side slope length and slope.

The MUSLE can be used to determine single event erosion losses. Most soil erosion occurs from a few severe storms each year. Therefore, by determining erosion losses for a few storms with different return periods an estimate of the total loss during the life of the impoundment can be made.

Gully Erosion Prediction

A method for predicting the depth of incision, gully width and location of the maximum point of incision is presented in section III.2.4. This approach can be used to determine the probable life of a cover. If the life is inadequate a maintenance plan can be developed or remedial actions defined. The method is simple to use but needs to be verified in the field. One limitation of the method is that it can only be applied to periods of not more than 200 years. For a design life of 1000 years it would be necessary to assume that the depth of incision would be 5 times that produced during 200 years or that corrective maintenance will be conducted every 200 years.

Embankment and Slope Stabilization Using Rip Rap

The permissible tractive force method and the critical shear stress method which were discussed briefly in section III.2.5 are commonly used approaches to design erodible channels. If the available materials are inadequate then it is necessary to either modify the geometry of the design or to provide a larger diameter cover material. Natural coarse noncohesive materials which are used for this purpose are known as rock riprap or simply as riprap.

Factors to consider when designing riprap are:

Riprap Characteristics

1) rock durability

2) density

3) size and shape

4) angularity and angle of repose

Flow and Slope or Channel Characteristics

1) flow depth and velocity

2) flow direction

3) bed and bank slopes

The design requirements for placing riprap on a cover vary depending upon cover location. It is suggested that the four areas of concern exist, as illustrated in Figure III.2-16.

Zone 1 is a zone of frequent saturation and occurs at the toe of the slope. Riprap must be sized to protect the toe from flooding in the major watershed and from flows discharging down the slope.

Zone 2 might be saturated occasionally and remains in the flood plain of the PMF. Riprap sizing criteria will be similar to that for Zone 1.

Zone 3 will seldom be saturated and is out of the flood plain of the PMF. In addition to flows associated with rainfall falling on the tailings impoundment side slope, it may be necessary to size the riprap for tributary runoff and flows from the cap.

Zone 4 is located on the cap and riprap protection will seldom be necessary. The diversion of upstream runoff and the collection and control of cap runoff is usually a more cost effective approach for Zone 4.

It can be seen that the riprap design requirements will vary from zone to zone due to changes in flows, geometries and degrees of saturation.

Numerous design procedures are available for selecting riprap sizes. In addition to the two methods discussed, commonly used procedures include:

1) Safety Factors Method (Simons and Senturk, 1977)

2) Stephenson Method (Stephenson, 1979)

3) U.S. Corps of Engineers Method (1970, 1971)

4) U.S. Bureau of Reclamation (USBR, 1978)

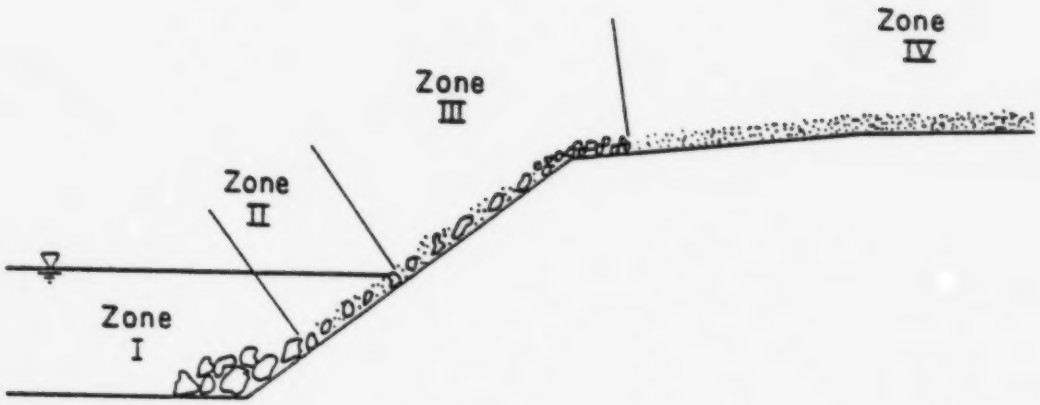


FIGURE III.2-16 ZONES OF A RECLAIMED IMPOUNDMENT REQUIRING RIPRAP PROTECTION

Reference should be made to the cited publications for details on the methods or to any modern text on open channel hydrology. Guidelines are suggested for the use of rock rip rap to prevent gully erosion by Walters and Skaggs (1986).

III.2.7 DIRECT VEGETATION TECHNOLOGY

Despite the knowledge gained during the past 15 years on the multiplicity of factors governing the reclamation of both acidic and non-acidic tailings no single and universally applicable technology has emerged which will adequately determine the reclamation potential of a closed-out tailings disposal area. Every such area requires individual assessment using a variety of assessment techniques to determine reclamation potential (Watkin and Watkin, 1982).

Several general principles have, however, been established for consideration when the time has come to reclaim a closed-out tailings disposal area. These include:

(i) The recognition that a high degree of scientific and technical expertise is required to ensure a successful reclamation program.

(ii) The need to form a reclamation team comprised of personnel with expertise in agronomy, engineering, hydrology, milling and water quality. Such expertise may be drawn variously from federal and provincial government agencies, independent consulting companies, universities and individuals of the mining companies concerned.

(iii) Realization that a reclamation program is comprised of both physical and biological facets and, in particular, that the latter facet cannot be designed to a foolproof level and may be subject to unpredictable failure compared to many of the physical facets.

(iv) To allow at least two years, preferably longer, for the development and implementation of a specific tailings disposal area reclamation program in order to minimize potential failure and to ensure the most cost efficient selection of reclamation techniques.

For the Canadian mining industry, reclamation undertaken by the majority of mining companies (the aggregate industry is excluded) veers towards a simplistic concept of reclamation - the establishment of a permanent self-sustaining and maintenance-free vegetation community on a tailings disposal area designed and constructed to provide permanent stability against failure of the tailings dam slopes. The objectives in establishing a vegetation community on a tailings disposal area are several-fold:

(i) Control of fugitive dust,

(ii) Erosion control to prevent gullying, sheeting and piping of tailings by water which could ultimately lead to failure of dam slopes,

(iii) Prevent the run-off of surface particles and subsequent sediment disposition in water courses,

(iv) A reduction in water volume seeping through acid and heavy metal contaminated tailings.

(v) Aesthetic improvements of tailings disposal areas, and

(vi) Acceleration of natural succession, i.e. colonization by indigenous plants from the environment surrounding, on tailings disposal areas.

In the case of uranium tailings disposal areas, achievement of most of the above objectives would also provide a control on the disposition of several radionuclides from the disposal area to the adjacent environment.

Direct Seeding vs Use of Covers

Vegetation may be established on tailings disposal areas using one of two general methods; firstly, the spreading of an overburden or topsoil cover over the tailings followed by seeding into the cover material or secondly, seeding directly into the tailings material itself. Both methods have their advantages and disadvantages.

In reviewing the advantages and disadvantages of topsoil/overburden placement on surfaces often considered to have a poor plant growth support potential Brown, (1982) questioned the commonly accepted premise that topsoil/overburden placement is a prerequisite for successful revegetation in such circumstances. Based on experience gained in high alpine areas of Colorado, Brown pointed out that adequate revegetation requires a satisfactory plant growth medium but that the medium need not be topsoil or overburden. Under most circumstances, topsoil/overburden is simply an amendment which accelerates the process of reclamation. There is ample evidence available from many tailings reclamation programs carried out in Canada that numerous types of tailings (e.g. uranium, base metal, precious metal and others) can be amended directly to form suitable plant growth media (Peters, 1978; Watkin and Watkin, 1982) on which permanent self-sustaining and maintenance-free vegetation can be established.

Unequivocal support for topsoil/overburden placement cannot be found in world land reclamation literature; indeed, except in cases of hazardous or polluted wastes, an argument can be developed from available data to support the contention for direct treatment and seeding of many disturbed areas rather than attempting to cover them with topsoil or overburden (including rock wastes). Among the most common objections to the use of topsoil/overburden materials are those of cost of placement, instability when placed on slopes and the possibility of contamination by upward leaching of acidity and trace metals from tailings having such contamination.

In general though, topsoil/overburden use is considered beneficial when:

(i) it can be stripped, relocated and respread immediately or within several months and not stockpiled for periods of one or more years,

(ii) it can be considered a suitable source of organic matter and/or fine soil particles on sites deficient in organic matter or largely devoid of fine soil particles, or if

(iii) the land being reclaimed is required for agricultural or intensive forest production, or

(iv) it is used as a cover or sealant over hazardous, toxic or polluted wastes.

One aspect of the use of topsoil/overburden materials that is rarely given sufficient consideration is that of the qualitative nature of the materials as plant growth medium. Indeed, as has been pointed out by Bradshaw and Chadwick (1980) there are numerous examples where the plant growth potential of topsoil/overburden materials is often less than that of the waste materials they are being used to cover. At one Canadian uranium mining operation six local soil materials were being considered for use as covers on tailings. A laboratory investigation (Mine Waste Reclamation, 1982) revealed that two of the six materials were vastly inferior to the other four in terms of ability to support plant growth (Table III.2-7). The differential between the two sets of cover materials was thought not determinable by chemical soil testing.

The tendency among those concerned with land reclamation to over-emphasize the plant growth potential of soil/overburden covers is an unfortunate one. As other studies have also indicated (Watkin and Watkin, 1984) there is considerable variability between soil/overburden materials. Some materials, which from chemical soil testing alone, would appear to be unsuitable as plant growth media, can with only relatively minor amendment be transformed into acceptable cover materials (Table III.2-8 - Site 32). Other materials, seemingly acceptable as cover materials, when only above ground plant growth is observed, may be subsequently rejected for use because they cannot sustain acceptable levels of root growth (Table III.2-8 - Site 34). By comparison a potential cover material at another mining property (Table III.2-8 - Site 33) which contained no inherent factors antagonistic to plant growth required only fertilization to ensure adequate levels of plant growth.

Hanney et al (1980) list three categories of "soil like" materials as potential covers for use at uranium mine sites in Canada: glacial till cover, clay cover and gravel or rip-rap cover. As discussed above in this section, an indiscriminate recommendation should not be made for the use of either glacial till or clay covers without first considering their potential to support a satisfactory level of plant cover. There are, of course, other considerations to be taken into account when recommending the use of glacial till or clay cover materials. For example, Hanney et al (1980) states that clay covers reduce radon emissions and water infiltration much more effectively than all other natural material covers; but clay covers are more susceptible to wind and water erosion and should thus be covered with gravel, rip-rap or waste rock. From an agronomic viewpoint, susceptibility to wind and water erosion can be satisfactorily controlled by the establishment of plant cover thereon and the use of gravel, rip-rap or waste rock would not be required, provided of course the particular clay cover in question has been sufficiently well evaluated to determine its plant growth support potential.

TABLE III.2-7

PLANT GROWTH EVALUATION OF SIX TOPSOIL/OVERBURDEN SAMPLES FROM AMOK LTD., CLUFF MINE, SASKATCHEWAN AND STANDARD AGRICULTURAL SOIL TEST ANALYSIS OF THE SIX SAMPLES BEFORE EVALUATION

	Plant Growth (Grams dry Weight) 71 days after seeding			Standard Agricultural Soil Test of Six Samples as Received and Before Addition of Fertilizer Regimes				
Topsoil/ Overburden Sample No.	Fertilizer Regime FR1	FR2	FR3	pH	Total salts umhos/cm	Phosphorous ppm	Potassium ppm	Calcium ppm
1	0.22	0.69	0.37	5.8	900	13	36	1550
2	0.35	1.08	0.67	7.9	500	12	116	1975
3	3.54	3.83	4.46	6.3	300	4	36	450
4	2.94	2.87	3.93	6.7	200	5	44	600
5	5.97	5.29	6.81	6.0	150	3	56	2000
6	4.31	5.09	4.32	5.8	100	4	44	325

Source: Mine Waste Reclamation 1982

TABLE III.2-8

VARIATION IN PLANT GROWTH* (LEGUME-GRASS MIXTURE) ON SOILS AND SUBSOILS AT 4 SITES IN EASTERN CANADA

	SITE 32			SITE 33			SITE 34		
		Limestone t/ac			Limestone t/ac				
	pH**	0	5	pH**	0	1	pH**	Fertilizer	Root depth
Soil No. 1	2.7	0.24	1.02	5.9	0.46	0.47	7.8	0.18	4 in.
2	4.8	0.89	0.85	5.8	0.29	0.37	7.8	0.26	2 in.
3				5.8	0.61	0.51	7.2	0.21	1 in.
4				6.0	0.45	0.40	7.2	0.24	2 in.

* grams dry weight per replicate
** bulk sample mean per soil

Source: Watkin and Watkin, 1984.

One very common problem arising from the use of cover materials during the reclamation of tailings or other waste disposal sites (e.g. sanitary landfills) is the difference in degree of compaction standards between engineering and agronomic requirements. Most often the compaction standard required by engineering concerns to provide cover stability or prevent infiltration of water is antagonistic to plant growth, particularly the development of root volume and depth of root penetration, two aspects of extreme importance in determining the long term persistence and development of plant communities.

Revegetation studies were begun by Rio Algom on inactive tailings areas in 1970. Since that time there have been numerous reports prepared and papers published describing various aspects of revegetation studies in the Elliot Lake region (e.g. Dunn et al, 1977); Watkin and Winch, 1973; Murray, 1977; Murray and Moffet, 1977; Murray, Webber and LaRocque, 1977; Mine Waste Reclamation 1980a, 1980b, 1980c, 1980d, 1981; Kalin, 1983). Similar revegetation studies were begun elsewhere in Canada at the same time also on tailings areas characterized by the presence of iron sulphides (e.g. Brooks, 1981; Gardiner, 1977). During the period 1970 to 1978 approximately, the majority of the studies were concerned with finding solutions to overcoming the acidic nature of the various tailings disposal areas and which would permit the implementation of large scale field reclamation programs. Because of that, the major proportion of vegetation established during the 1970-1978 period was in the form of numerous experimental areas rather than large area blocks.

As a result of the above experimental approach, considerable variation occurred between the various experimental areas in the amount of vegetation that was established. Many experimental treatments were either not successful or were only partially so; partly because inappropriate neutralizing agents or non-adapted plants were utilized or, in other cases, because insufficient levels of application of appropriate neutralizing agents or fertilizers. An understanding of the reasons for such variability in revegetative cover, as observed between 1980 and 1985, is extremely important in assessing the role of vegetation as a cover for uranium tailings.

Much of the investigative reclamation work in the Elliot Lake mining camp was conducted at the Lacnor and Nordic closed-out tailings disposal areas of Rio Algom. Because most of the level areas at these two sites have now been subject to reclamation inputs, both experimental and larger-scale, they have been considered major examples of revegetation practice and thus considered demonstrative of the role of vegetation as a cover for uranium tailings elsewhere in the Elliot Lake region. Certainly a sufficient vegetative cover has been established at both sites to provide at least in the short term, adequate control of wind and water erosion. Whether or not, the present degree of vegetation establishment is sufficient to provide a measure of long-term stability (measured in many decades) and to ensure that natural ecological succession will occur to the extent that the two tailings areas will eventually support a forest cover typical of the Elliot Lake region, is currently the subject of some debate (e.g. Kalin, 1983, 1984).

It is unfortunate that various reviews or observations on the quality of vegetation establishment on uranium tailings have not given sufficient consideration to possible reasons why certain areas or sections of a particular tailings area (e.g. Lacnor and/or Nordic) do not have an acceptable level of plant cover. For example, one reviewer (McInnis, 1985-page 34) states that "revegetation attempts have resulted in patchy growth at best, and some of the growth has eventually died from acid-kill, drowning, drought, tailings windblast or a combination of these. At present, the vegetation on most of the tailings is an inadequate cover. It has a tenuous hold on survival". While it is not clear if McInnis is reporting on his own observations or presenting those of others, whatever the case, there is a strong tendency to ascribe vegetation failure to weaknesses within the vegetation itself and not to possible causes of failure from inadequate inputs by humans.

Acid-kill of vegetation in tailings areas where acidification of the tailings surface has occurred for several years is an indication that insufficient agricultural limestone has been incorporated into the tailings at time of revegetation. Studies by Mine Waste Reclamation in 1981 (Mine Waste Reclamation, 1981) on Nordic and Lacnor tailings areas showed that different sections of each tailings area had different limestone requirements for neutralization of acid conditions prevailing. That variation in agricultural limestone requirements within any tailings area containing oxidizable iron sulphides is now well documented for a number of Canadian mines (Watkin and Watkin, 1982) and has very important practical implications for the long-term persistence of established vegetation.

The destruction of vegetation by drowning and tailings windblast is a consequence of reclamation program design and management and not an inherent failure of vegetation itself. Numerous older tailings disposal areas are subject to periodic flooding and prolonged periods of surface water inundation. At time of close-out of those areas, current reclamation practices were not a requirement; practices which now require that tailings disposal areas be closed out in a form which prevents flooding and surface ponding and ensures the most rapid surface run-off. Both the Lacnor and Nordic tailings disposal areas would be classified as belonging to the older classification. This is not to imply that flooding control practices have not been implemented at either property In fact, they have, and surface flooding events have been greatly reduced in number, but the older nature of those areas means that surface flooding control cannot be controlled with the precision and efficiency expected in new tailings disposal areas today.

The interpretation of vegetation destruction by drought has also to be carefully considered. While drought can be a true causative factor in the Elliot Lake region, there are also many instances where it is a secondary, not primary factor. Vegetative growth which is under stress due to inadequate fertilization practices, inadequate acid neutralization practices or to seeded species being only marginally adapted to the "soil" conditions pertaining in the tailings area is more likely to succumb to drought than vegetation which is not under stress due to the above stressful factors.

Kalin (1984) is quoted by McInnis (1985 - page 32) as saying that trees of the dryland environments are growing at their limits of ecological tolerance. That point of view is considered to be too premature in terms of implying that future vegetation development on uranium tailings will not result in the development of forest-type vegetative cover exhibiting long-term stability. It must be remembered that the natural colonization by indigenous species on tailings areas in the Elliot Lake region is very much a reflection of the experimental agronomic practices undertaken to determine the most rapid, and least expensive, means of establishing a grass or grass-legume plant cover on the tailings in order to control water and wind erosion to as a great a degree as possible.

A recent paper (Lawson, 1986) reviewed various options for reclaiming the Gunnar uranium tailings in Saskatchewan. The options included use of vegetation, rock covers and engineered structures to control wind and water erosion in situ and potential disposal of tailing to adjacent land and water environments. A number of misconceptions are evident with regard to revegetation.

For example, it is stated that it is useful to classify the tailings at Gunnar Lake into areas based on slope (flat, inclined), moisture (moist, dry), salinity (saline, non-saline), acidity (moderate, slight), depth to water table (cohesive, loose) as these parameters relate to vegetation cover (vegetated, barren) and erosion potential (water, wind). That broad classification was used to delineate five main surface units on the Gunnar main tailings disposal area which were: a windblown area, vegetated area, deposition area, water covered area and a floodplain area. Such a broad classification scheme and delineation of surface units could equally well be applied to nearly all other tailings disposal areas in Canada and other countries around the world. Tailings reclamation experience in Canada and those other countries (Bradshaw and Chadwick 1980; Marshall, 1982; Watkin and Watkin, 1982; Rummery and Howes, 1978; Chamber of Mines South Africa, 1979) has clearly indicated that consideration alone of physical and chemical characteristics of tailings is totally inadequate in determining the revegetation potential of tailings disposal areas; for example;

(i) In slightly acid, neutral or slightly alkaline tailings the major factor in preventing natural colonization by indigenous species or the establishment of agronomic species is inadequate plant nutrition. This is due to the absence of/or inadequate levels of, the macronutrients nitrogen, phosphorous and potassium in the tailings. In this instance indigenous or agronomic species cannot establish rapidly enough (i.e., rate of germination, rate of root elongation) due to the effect of wind erosion such as surface particle movement, sandblasting of newly emerged seedlings and surface water evaporation. Although wind erosion effects are critical in terms of plant establishment, they are secondary causes of failure and do not represent an inherent inability of the tailings in question to support a vegetative community.

(ii) On certain tailings disposal areas, the effect of the absence of macronutrients can be compounded by the presence of excess acidity or alkalinity in the tailings. In order to establish an acceptable plant cover the excess acidity or alkalinity must be controlled by

the addition of one or more amendments which will eliminate or reduce the excess factors. An adequate plant nutrition program must then be applied.

(iii) Depth to water table and texture of tailings are rarely primary factors in preventing vegetation establishment except where:

- the water table is sufficiently close to the tailings surface that the tailings surface is permanently saturated and root growth is inhibited by low concentrations of oxygen and

- where tailings particle size is of such coarseness (e.g., china clay tailings - Bradshaw and Chadwick, 1980; lead-zinc smelter slags - Mine Waste Reclamation, 1982) that insufficient plant available water is retained in the surface three inches or thereabouts to enable germination and root elongation to occur.

In both situations (saturation and excessive surface drainage) certain reclamation procedures can be applied which will remove these barriers to vegetation establishment. The barriers do not in themselves represent an inherent property of the tailings against vegetation establishment.

It must be recognized that certain tailings do contain inherent properties that provide insurmountable barriers to vegetation establishment. These properties are determined by the mineralogical characteristics of the ore from which the tailings originated and include the presence of phytotoxic concentrations of one or more trace metals, an excessively large acid generating capacity due to the presence of iron sulphides or the presence of excessive salinity caused by dissolution of various minerals present in the tailings.

Salt generation, accumulation and movement in tailings, is from a vegetational viewpoint, a poorly studied and poorly understood factor. Generally, salt content of tailings is measured as in agricultural soils and expressed as electrical conductivity or specific conductance. Interpretation of salt concentration follows the parameters and understanding developed for agricultural soils in the western United States and Canada, which is primarily based on standards developed by the United States Department of Agriculture (Richards, 1950). Where the soluble salt content of a soil exceeds 4.0 mmhos/cm the growth of many agricultural crops is moderately to severely restricted. In agricultural soils where the dominant soluble salt is sodium, another parameter, the sodium absorption ratio (SAR) is used as an assessment method for predicting plant growth on such salt affected soils.

Sodium salts are not considered a major factor in affecting vegetation growth on tailings in Canada. Salts found in acidic tailings are generally sulfates. Water percolating through acidic tailings containing pyrite will contain one or more of the sulfates of magnesium, aluminum, iron and calcium, depending upon the mineral composition of the tailings. Salt concentrations will, or can, vary considerably in the surface layers of a tailings disposal area, depending upon the degree of mineral separation

that occurs as the tailings are laid down, and commensurately the pyrite oxidation rate and magnitude. Where carbonate minerals are present in the tailings, the effect of these on the end products of the pyrite oxidation process can be very different, and generally beneficial to future vegetation establishment programs.

Upward movement of salts in tailings and deposition on the tailings surface as clearly visible salt pans is a common phenomenon during periods of hot, dry weather. The salt pans are redissolved by subsequent precipitation and washed into the tailings profile. Practical experience has shown (Watkin 1986 personnel communication), that unless relatively high levels of salts are present in the tailings (> 2 mmhos/cm) upward movement of salts is controlled by the establishment of a healthy and vigorous herbaceous vegetation. In numerous instances where vegetation failure has been attributed to upward salt movement, subsequent investigations have indicated that a poor vegetation cover (per cent ground cover and low plant vigour) was present due to inadequate site preparation, insufficient tailings amendments, incorrect fertilization practices and plant species selection). Unfortunately, the above comments are based on subjective evaluation and conclusions and no real documentary evidence has been obtained to adequately verify these conclusions.

Frequently, the phenomenon of upward salt movement in non-vegetated tailings has been considered a major problem in tailings reclamation programs; a phenomenon that can only be resolved by the placement of a two layered capping system on the tailings surface. The first layer that is placed directly onto the tailings surface consists of a layer of coarse gravel or similar waste material that interrupts the upward capillary movement of salt affected water. That break-layer is then covered with a topsoil and/or overburden layer in which vegetation is established. The necessity for a two-layered capping system to present upward salt movement is much more probable under arid or semi-arid climatic environments characteristic of regions in the western United States, (e.g., Sandoval and Gould, 1978; Schuman and Power, 1981) and Australia (Longworth, 1977). A requirement for a two-layered capping system under the more temperate climatic conditions pertaining to the Elliot Lake and northern Saskatchewan regions cannot yet be adequately resolved for many of the reasons discussed earlier. Michelutti (1978) concluded that the use of a two-layered capping system was the most promising method of achieving the primary objective of a maintenance free vegetation cover on tailings containing a high percentage of pyrrhotite. The ideal overburden was found to be 8 cm of coarse material (0.6 cm gravel) overlain with 8 cm of a loam soil. It may be suggested that previous revegetation studies attempting to directly revegetate the tailings were incomplete in several aspects, but the investigation which recommended the use of a two-layered capping system did result in the achievement of a successful tailings revegetation program.

Plants vary in their sensitivity to soluble salts. Acid, pyritic tailings are difficult to revegetate since the acid component alone can be sufficient to inhibit plant growth. Treatment of acidic tailings with agricultural limestone causes those salts whose solubility is dependent upon low pH to precipitate. At a pH of 6 to 7 the predominant soluble salt in tailings would be calcium sulphate, and in numerous circumstances, magnesium sulfate, particularly where a dolomitic limestone has been used

for acid neutralization. Magnesium sulfate is the greater problem of the two salts because of its greater solubility.

The role of excess salts in soils in affecting plant growth and vegetational community development on agricultural soils is well documented in many soil science textbooks (e.g., Brady, 1974; Russell, 1964) but is entirely confined to non-acidic salt-affected soils. More recently, fairly extensive studies have been conducted and reviewed on chemical processes occurring in acid sulphate soils and pyrite-bearing coal spoils (Evangelou, Phillips and Shepard, 1982; Evangelou and Phillips, 1984) but similar type investigations have not been undertaken for tailings arising for hard rock mining operations. The extrapolation of knowledge gained from agricultural soils and coal mine spoils is fraught with difficulty. Other soil chemical characteristics applicable to tailings are known to be different, and behave differently, compared with agricultural soils and coal mine spoils (Rowell, 1981; Watkin and Watkin, 1984; Berg, 1978). It is reasonable to expect that soluble salt aspects are not an exception. Nielsen and Peterson (1978) have suggested that the rating scale for soluble salt content and crop tolerance developed for agricultural soils by Richards (1954) are likely to be too high for use on evaluating tailings materials where soil-water-plant relations are less desirable. In that respect, Watkin (1986, personal communication) has noted from revegetation investigations on a wide variety of base metal and uranium tailings, that a measured salt content exceeding 2 mmhos/cm is indicative of potential plant growth problems irrespective of the acidity (pH) of the tailings in question.

In assessing the reclamation potential of tailings disposal areas it is imperative that a true distinction be made between those inherent tailings properties which basically determine the revegetation potential of tailings (and which arise from their mineralogical composition) and the "apparent" properties. The "apparent" properties are those which are determined by the method of tailings disposition (to form dry or wet disposal areas), and exposure to climatic influences (wind and surface run-off). It has been suggested that the former set of properties be classified, in revegetation terms, as the biological potential of tailings to support a vegetative cover, while the latter set of properties or characteristics be termed the practical potential.

III.2.8 SOIL COVER

From a revegetation viewpoint, cover materials placed on the flatter portions of closed-out tailings areas may be considered stable once an acceptable level of vegetation has been established which will control wind and water erosion.

The major problem associated with cover placement on tailings areas is that of obtaining a uniform final grade or contour sufficient to prevent surface ponding of spring melt and rainfall.

In practical terms, the deeper the depth of cover placed, the easier it is to obtain the final uniform grade. A minimum depth of cover of 46 cm (18 inches) may be required to obtain the desired grading when large areas of placement are involved (e.g. 10 ha (25 acres)) but the practical

situation may be confounded by a number of factors. A final tailings surface is not necessarily level when measured on a microtopographic scale. The effect of wind erosion and surface run-off can cause surface irregularities in microtopography where elevation differences ranging between 8 cm (3 inches) to 16 cm (6 inches) can occur many times over relatively short distances. While such elevation changes are minor over a tailings disposal area as whole, they are of extreme importance in determining the quantity of cover material required to form a final uniformly graded cover surface. More extreme differences in elevation often result from the formation of gullies and rills.

Ideally, a tailings surface should be graded to remove the above described elevational differences prior to the placement of cover materials, but frequently that cannot be achieved due to poor trafficability on the tailings caused by a high water table. In such situations, cover placement is restricted to wintertime when the tailings surface is frozen and permits access by vehicles. Where only 30 cm (12 inches) of cover is intended to be placed, the resultant depth of cover material tends to be variable and a satisfactory final cover or cap is not achieved. One result of that variability in cover depth, particularly where the underlying tailings is not suitable for revegetation, is that any vegetative cover established on the cover, will itself be variable.

Apart from revegetation concerns, there are a number of factors to be considered in what type of cover material (i.e. glacial till or clay or rip-rap) should be selected. These are considered in detail elsewhere in this report. In terms of establishing a vegetative cover on the placed materials it may be generally considered that a clay cover is probably not desirable. Apart from practical difficulties encountered in the spreading and grading of clays to achieve a desired final contour, both immediate and longer term difficulties are likely to occur in vegetation establishment and persistence, respectively. In general, clay covers are liable to maintain a plastic condition for a longer period time following snow melt and spring run-off than covers composed of glacial till materials. That condition delays the accessibility of reclamation equipment and preparation of a seedbed and subsequent seeding sometimes to the extent that the available plant growing season in the year of establishment is very much reduced. Under the climatic conditions pertaining in northern Ontario and northern Saskatchewan, it is recognized that any shortening of the available plant growing season in the establishment year may have dire effects on achieving a satisfactory level of plant establishment. For example, on a copper tailings disposal area in northwestern Quebec where a clay cover was placed over the highly acidic tailings, the establishment of a vegetative cover was delayed by over one year because reclamation contracting equipment could not gain access to prepare a satisfactory surface for seeding (B. Brooks - personal communication) in late spring or early summer. Operation of reclamation equipment on wet clay covers can lead to serious compaction, puddling and structural breakdown of the clay cover which will inevitably have adverse effects on the aim of establishing a satisfactory degree of vegetative cover. In the longer term, the phenomenon of cracking of clay covers during extended periods of drought and the associated "hardening" of clay covers during such periods is destructive of vegetative cover and may result in increased erosion should the drought periods be ended by severe rainstorms.

Where cover materials are placed on dam embankments the potential for erosion is increased. Topsoil/overburden slippage on slopes can be a severe deterrent to their use, since in such situations, the roots of vegetation established on the cover may remain confined to the cover material and not cross the interface between the cover material and the underlying tailings. Hence, there is no binding occurring between the cover material and the tailings. Where the topsoil/overburden becomes saturated during heavy precipitation periods, that lack of binding may often result in the slippage of the topsoil/overburden, destruction of the vegetative cover and create potential new sources of erosion unless the disturbed area is repaired and reseeded.

III.2.9 WASTE ROCK

Three major options have been stated as being available for the stabilization of uranium mill tailings: (1) rock cover, (2) soil and revegetation, or (3) a combination of both on different positions of the tailings area (Voorhees et al, 1983). The optimal choice among these alternatives depends upon site-specific characteristics such as climate, local geomorphology and soils and on design variables such as embankment height and slopes, depth of cover material, modification of upstream drainage and revegetation practices.

Basic design requirements for erosion control have been reviewed by Beedlow and Parker (1985) with special reference to uranium property closeouts in the western United States. The basic concepts are, however, equally applicable to Canadian situations. The basic elements of a protective cover system are to control potentially disruptive erosive forces such as surface run-off and rainfall impact, flooding, wind, mass wasting, geomorphic hazards and biotic intrusion. Erosive protection requirements will vary according according to expected or calculated erosion stresses. The degree of protection ranges from that provided by established vegetation (Beedlow, 1984) to that of heavy duty rip-rap (Walters, 1982). Beedlow (1984) proposed that vegetation should be established at all sites (in Western U.S.A.) to prevent the accumulation of excess soil moisture when rock covers are used for surface stabilization. The design of riprap waste rock covers has been reviewed in sections III.2.5 and III.2.6. Waste rock and/or rip-rap will be utilized to varying degrees at several uranium mining operations in Canada (Feasby, 1985; Filion, 1985; MacLaren Engineers and Geocon, 1984 - quoted by McInnis, 1985), and the revegetation of waste rock environments is discussed below.

The revegetation of waste rock environments by direct seeding or planting of grasses, herbs, shrubs and trees has often been a subject of considerable debate among land reclamationists. Whether level surfaces or steep slopes, an environment composed of boulders, stones and few soil-size particles, subject to excessive drainage and often of poor nutrient status, has been considered too inhospitable to support satisfactory levels of plant growth. Surface modification, primarily spreading of overburden or topsoil materials has often been advocated as necessary prior to revegetation. There is now, however, a body of experience gained in Canada and other countries that not all rock-composed surfaces represent an environment incapable of supporting plant growth. Various studies have

shown that provided agricultural and commercial forest levels of plant productivity are not expected, grass, herbaceous, shrubs and tree species can be established directly, or enabled to naturally colonize rock surfaces.

Rock slopes classified as skeletal or fragmental can support a forest cover (USDA - Soil Conservation Service, 1973). Plant colonization and succession on high attitude talus slopes lying at the physical angle of repose was described by Leach in 1930. Bradshaw and Chadwick (1980) and Humphries (1979) have reported that rock wastes exhibiting slope angles of 35 degrees can be successfully revegetated with herbaceous and forest species. Extensive and successful revegetation studies have also been undertaken in Norway (Norsk Hydro, 1972) and in Colorado (Burns, 1980). Until 1980 approximately, studies on slope stability and revegetation procedures were largely limited to those undertaken on coal mine spoils in western Canada. Murray (1977) pointed out that little work had been done in Canada on seeding hard rock mining waste rock dumps or on the design and planning of such dumps with respect to revegetation. In 1986, it appears correct to say that there have been no integrated scientific studies of surface movement in waste rock dumps and rip-rapped slopes between the physical and biological angles of repose. The physics of surface movement in relation to waste rock type, particle size, weathering characteristics, internal and surface drainage and potential effects of plant growth on both short term and long term stability are largely unknown for waste rock surfaces of the magnitude represented by numerous examples in the Canadian hard rock mining industry.

Waste Rock Reclamation

Specific Aspects

Three main aspects of a waste rock substrata are considered very important in vegetation establishment schemes:

(i) macrostability or freedom from risk of deep seated failures,

(ii) surface stability or freedom from continuing surface movement or erosion which would prevent vegetation establishment, and

(iii) physical and chemical properties of the substrata in relation to plant growth requirement.

Given that the first two aspects can be resolved through engineering design of tailings impoundments or waste rock disposal areas, the physical and chemical properties of the substrata become critical factors when establishing a vegetation cover.

Visual observations of waste rock disposal areas, and slope areas where rock waste has been placed over other substrata as a means of erosion control at many mines in Canada show that a tremendous variability exists in waste rock particle size both in a vertical and in a horizontal direction. Three main particle size surface categories may be arbitrarily defined (Watkin, 1985):

(i) Massive to small boulders with no interspersed fine (soil size) particles and large interstices occurring to depth.

(ii) Massive to small boulders, interspersed with coarse and fine (soil size) particles at or below surface; interstices are small or occupied with fine particles.

(iii) No massive or small boulders, surfaces composed of small stones interspersed with coarse and fine particles (soil size); interstices are small or occupied with fine particles. Surface movement (wind and water erosion) could often be problematical.

Each of the above three categories will require different surface treatment if a vegetation cover is to be established. In the first category the spreading of an overburden or topsoil material would be necessary in order to provide the semblance of a plant growth medium. Practical experience has, however, shown that the results of spreading topsoil on overburden materials, particularly those which are clay-like, can result in plant growth and persistence problems during extended drought periods. The topsoil/overburden materials tend to remain suspended on, or near the surface of the waste rock pile, and not to penetrate into the rock pile. Consequently, plant roots are restricted to the cover material and are dependent upon sufficient plant-available water occurring in the suspended material at all times. Because of the lack of a suitable watering medium at depth, the normal characteristics of rooting systems of growing to depth to ensure water availability cannot take place. In periods of extended drought, therefore, the "suspended" root and shoot mass undergoes dehydration, often resulting in death of the plants.

Rock wastes classified in the second category can be vegetated by hydroseeding an appropriate seed mixture - fertilizer - mulch combination onto the rock waste surface. Practical experience has shown that vegetation establishment is greatly enhanced if, after applying the above combination, a second application of water only is applied to the rock wastes surface in order to wash the various constituents into surface crevices.

By contrast, the potential surface instability of rock wastes classified in the third category would have to be alleviated prior to seeding. That may entail slope regrading, placement of additional non-erodible materials, construction of diversion channels above or on the slope itself to control surface-runoff or increased inputs of mulch and soil stabilizer during the seeding process.

Under field conditions, two or three of the above rock surface categories often occur at any one site or area. Where rock waste has been simply dumped, separation of variously sized particles will occur as it falls down the slope, with the finer particles separating out at the top of the slope and the coarser materials passing further down the slope. This will often mean in practical revegetation terms that a particular slope will have to receive more than one type of seeding procedure when being reclaimed.

There is a strong tendency in the literature describing the use of rock waste, rip-rap or gravel covers as erosion control measures on tailings

impoundments to assume that revegetation of such covers will be relatively straightforward. It has been stated that revegetation technology is available from the mining and construction industry in general and can be applied to uranium mill tailings with little effort (Beedlow, 1984). While such conclusions have been made primarily with reference to the western United States, it has been noted that similar assumptions have been incorporated into mine reclamation plans for a number of mining properties in Canada including uranium mining operations. Such tendencies and assumptions are considered to present a far too simplistic situation as far as the uranium mining industry in Canada is concerned. There are two major reasons for doubting the relative ease which has been assumed in the revegetation of rock-based covers at some future point in time.

Based on Canadian experience, the successful revegetation of rock wastes, rip-rap and ground covers has been almost entirely confined to relatively small areas and or structures (e.g. Mine Waste Reclamation 1982, 1985) where sloped embankments are concerned (approximately 9 metres), though considerably more success has been achieved on level rock waste areas (McArthur, 1985). In general, where rock waste faces exceed 15 metres, considerable difficulties are likely to be encountered in the establishment of a vegetative cover by direct seeding on such slopes. The difficulties arise in some instances because of the variability in the rock waste surface as a plant growth media and in the technical difficulties in applying seeds mixtures and associated components to those surface areas. Presently, one Canadian mining property (Hamaguchi and McArthur, 1985) is undertaking a large scale investigation program to develop the technology for revegetation of waste rock covered areas which occur at the property.

As well as technological and physical characteristic problems associated with rock waste covers on slopes, consideration must also be given to those rock waste covers which are underlaid by pyritic tailings or which have been contaminated through wind and water erosion with pyritic tailings.

III.2.10 WEATHERING OF ARMOURING FOR TAILINGS IMPOUNDMENTS

The durability and weathering characteristics for armouring (rip-rap) for uranium tailings impoundments require assessments over the longer time frames than other types of waste deposits: e.g., typically 1000+ years, as against less than 100 years. Conventional techniques for the assessment of the durability of materials may not be applicable for these longer time periods, therefore.

Weathering of armouring can be considered in terms of both physical and chemical processes. A number of approaches or methodologies have been used to assess physical durability of rock for the purpose; however, all methods appear to be relative. Methods include:

- deduction from studies of overland erosion on surfaces in terms of rock size, mass and shape (Walters and Skaggs, 1986 is an example of the overload erosion type of study).

- implications from the survivability of ancient man made structures, such as burial mounds (Lindsey et al, 1983); and

- direct selection and testing methods (Foley et al, 1985).

None of these methods assess chemical weathering effects. While all methods are relative and comparative, the last method provides semi-quantification or ranking of materials and the chain-of-deduction required is not as known.

Foley et al suggest an initial (reconnaissance) phase and secondary (demonstration) phase in material evaluation and selection. The initial phase includes the use of geologic maps, aerial photographs and sampling to locate and select the most likely materials for use, based on their apparent durability and lack of erosion/weathering. Testing using slake-abrasion and wetting-drying tests is performed on collected samples. The follow-up, secondary phase involves a detailed site examination and analysis to attempt to include the effects of chemical weathering, also.

Chemical weathering and durability of materials to chemical effects are more difficult to quantify. Wetting-drying tests and slake abrasion tests can be performed, however, with simulated rainfall (water at pH 5.50) to include the potential for the mild, acidic leaching due to rainfall and surface wash. The ability of extrapolation of such test data for long time periods is questionable, however.

III.3.0 WIND EROSION

III.3.1 WIND EROSION MECHANISM

Soil erosion by wind may be expected wherever the soil is bare and finely divided, the surface is loose and dry, and the wind is strong. These conditions might occur during tailings deposition operations and in the first few years of reclamation. Once vegetation is well established, wind erosion should only be a factor if erosion due to runoff has occurred, or in the early spring when the vegetative cover is the least.

Problems encountered with wind erosion include:

1) loss of cover material;

2) atmospheric pollution;

3) the potential for uranium and radioactive daughter products to be transported great distances;

4) safety hazards due to reduced visibility.

Generally, wind erosion will be secondary to water erosion and measures taken to prevent water erosion will also control wind erosion. However, it should be noted that in the case of water erosion, downstream controls can be installed to prevent further sediment migration. Except in the immediate vicinity of the site, downwind measures to control wind erosion are ineffective and impractical.

The wind erosion cycle is illustrated in Figure III.3-1. Loosening of cover material could occur due to:

1) freezing and thawing;

2) wetting and drying;

3) raindrop impact;

4) flow down the side slopes.

Following loosening, movement of particles within the particle size range of 0.1 to 0.5 mm could occur. Particles larger than 0.5 mm might be transported a short distance but are sufficiently heavy to prevent long distance movement except in severe storms. Particles smaller than 0.1 mm do not protrude enough to be moved by wind. However, significant quantities could be entrained in severe turbulent wind storms. Transportation of particles will occur due to saltation, atmospheric suspension and surface creep.

Most erosion occurs due to saltation. The wind velocity at the cover surface is almost zero and soil particles receive a greater impact on their tops. A spin is produced and the top of the particle moves at a greater speed than the wind. Movement then occurs due to the horizontal force of the wind and the spinning of the particle. A partial vacuum is created

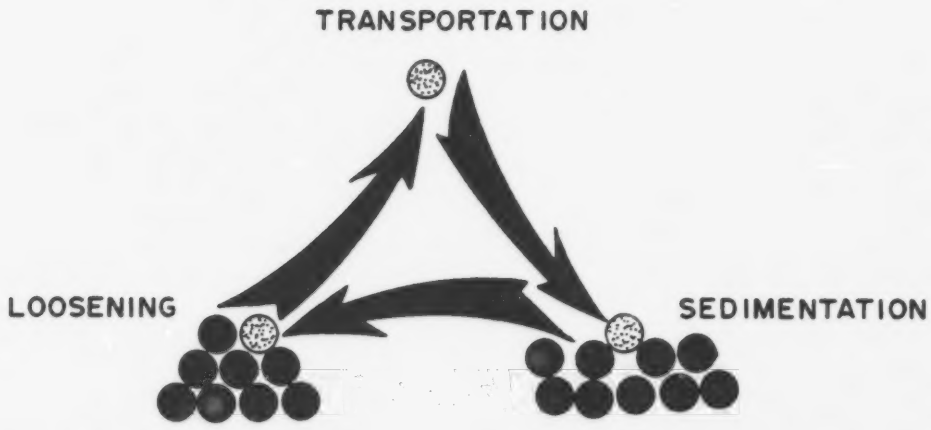


FIGURE III.3-1 WIND EROSION CYCLE

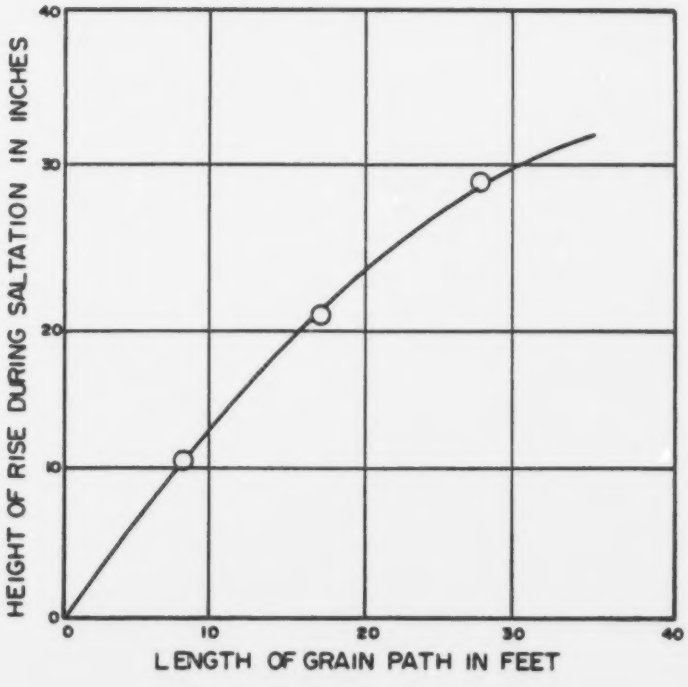


FIGURE III.3-2 HEIGHT OF RISE VS LENGTH OF SALTATING GRAIN PATH

above the particle, while the air is compressed on the underside. This causes the particle to jump, 15 to 30 cm, into the air. Only particles ranging in size from 0.1 to 0.5 mm in diameter can be transported by saltation. The height of rise versus the length of saltation is shown in Figure III.3-2.

Particles smaller than 0.1 mm will be transported by atmospheric suspension. Particles displaced by bouncing saltation particles are kicked into suspension and remain in suspension due to their small weight and relatively large surface area. Once in suspsension these particles can travel thousands of kilometers.

Soil particles larger than 0.5 mm in diameter are transported by surface creep. Particles roll or are pushed along the ground. Again, saltation plays a major part because when saltating particles strike the ground they assist with the movement of the larger particles. Particles larger than 3 mm in diameter are only transported by hurricanes and tornadoes.

It has been estimated that saltation produces 50 to 75 percent of the weight of wind erosion, atmospheric suspension produces 3 to 40 percent and surface creep 5 to 25 percent.

III.3.2 ESTIMATING WIND EROSION LOSSES

Very little wind erosion occurs in well vegetated areas. This is primarily because vegetation reduces wind velocities near the surface, increases surface roughness, and increases soil cohesion. Also, the soil moisture content in the root zone of a vegetated area will be higher than in a similar area with no vegetation. Moisture enhances cohesion and soil aggregation, and reduces the potential for wind erosion.

It might be expected therefore, that wind erosion losses could be predicted by an equation which considers soil erodibility, surface roughness, vegetation and climatic conditions. The most commonly used equation is (Chepil and Woodruff, 1963):

$$E=I*C*K*V*L \qquad (3.1)$$

where E is the soil loss in tons per acre per year, I is the soil wind erodibility factor, C is a total wind erosion climatic factor, K is the soil surface roughness factor, V is a vegetation factor, and L is the length of the unshielded distance parallel to the wind.

Tables, graphs and maps have been developed for determining the parameters in Equation 3.1 (USDA, 1968 and Israelsen et al., 1980). Parameter estimation is based on site specific soil and environmental factors such as:

1) mean wind velocities and predominant direction

2) wind fetch distance

3) exposure of slopes to wind

4) soil particle size distributions

5) cover material roughness

6) vegetative cover density

7) mean annual temperature and rainfall

III.3.3 PROTECTION OF COVER MATERIALS

Except in areas which are susceptible to hurricanes and tornadoes, wind erosion will not have a major impact on tailings impoundment cover materials. Rip-rapping, self-armouring of coarse material and the development of a vegetative cover will result in a significant reduction in the potential for wind erosion within a few years of reclamation.

Methods which are used to prevent or control wind erosion include:

1) installation of windbreaks

2) development of a vegetative cover

3) riprapping

4) soil stabilization

5) dust suppression through frequent watering

Riprapping and the development of a vegetative cover are measures which will be provided to control water erosion. However, through use of equations 2.15 and 3.1 total soil losses from wind and water can be estimated. If it is found that wind erosion losses are significant, then different plant species might be considered or large riprap selected.

Soil stabilization and dust suppression methods only provide short term benefits. In some instance such measures might be implemented during the establishment of vegetation but this will not be a general practice.

Windbreaks are either located upwind from a site to reduce wind velocities or downwind to trap transported dust particles. In terms of preserving tailings impoundment covers, only upwind windbreaks are of importance. The installation of windbreaks should always be considered because the main mechanism for wind erosion is near-surface wind velocity.

Windbreaks could provide benefits in a number of different ways, namely:

1) deflect winds around the site

2) deflect winds higher above the site

3) reduce wind velocities

4) increase soil moistures at the site

The effect of a windbreak will depend on the siting and physical characteristics of the barrier. If shrubs and trees are used the windbreak will probably only act as a partial barrier and wind velocities will be reduced. If synthetic materials are used they may serve as a complete barrier, thus forcing the wind high above or even around the site. In all cases an increase in soil moisture should be expected due to the reduced evaporation associated with the decrease in wind exchange.

III.3.4 CHEMICAL STABILIZATION

Wind erosion from both active and closed-out uranium tailings areas can represent a major, but sporadic, problem in the Elliot Lake and northern Saskatchewan mining camps. Short-term control (three months to three years) to prevent windblown tailings from being redeposited on areas outside the tailings impoundment may be achieved by spraying or spreading of a chemical which binds wind erodible size tailings particles: Permanent control is dependent upon covering the tailings impoundments with non-wind erodible wastes e.g. waste rock, or establishing a vegetative cover to reduce wind velocity at the tailings surface.

In any review of chemicals for wind erosion control, reference is usually made to two particular studies: those by Dean, Havens and Glantz (1974) concerning mine tailings stabilization and by Armbrust and Dickenson (1971) concerning agricultural soils. Both studies are now regarded as classic investigations in their particular fields. An earlier investigation (Havens and Dean 1969) dealt with chemical stabilization of uranium tailings at an Arizona mining property. A recent and much detailed review of chemicals for uranium tailings stabilization was undertaken by Elmore and Huntley (1984, 1985) with specific reference to a property in Wyoming. All of those studies strongly indicate that the efficiency of any particular chemical in controlling the windblowing of tailings is very site dependent and that there is no one chemical which is universally effective.

Following laboratory tests on 45 commercial chemical stabilizers in which wind tunnel studies were conducted to assess application rates, temperature (freeze-thaw) cycling and wet/dry cycling along with additional testing to determine the effects of ultraviolet light and water erosion on the durability of the stabilizers, Elmore and Hartley (1985) selected 17 chemical stabilizers for evaluation under field conditions. At the same time, field testing of eight commercially available windscreens was conducted. The data collected over a one year period were then developed into a model case study to compare the cost of the windscreen systems to chemical stabilization for a hypothetical inactive tailings pile.

The model study illustrated that the use of windscreen systems for wind erosion control may be more effective than chemical stabilization for an inactive tailings area. During the operational period of a tailings pond, however, chemical stabilization, or possibly a combined stabilizer/ windscreen system may be a more cost-effective approach.

In Ontario, wind erosion has generally taken place at each of the tailings impoundments where the sands are either not covered with water or capped. The most pronounced wind erosion has taken place at the Stanrock site.

Work done by Dave et al, (1985), indicate wind transported tailings have been found 1 km downwind of the site.

To combat against the wind's erosive nature a number of techniques have been used successfully in the area. These include the application of lime, bark or straw mulch to the surface, a complete revegetation program and/or flooding of the tailings.

In northern Saskatchewan, significant wind erosion has been reported at the Gunnar and Lorado sites. The upper portion of the Gunnar site is particularly unstable and wind transported tailings up to 1 m depth have been reported in natural vegetation around the tailings perimeter (Kalin, 1981). The Lorado tailings were reported by Swanson and Abouguendia (1981) as generally lacking natural vegetation and being subject to both wind and water erosion. The same authors reported that the only natural revegetation was some grasses (mainly reed grass) that was sparsely established where the tailings were thin, and some young birch trees which were invading the edges of the tailings.

Few other problems with wind erosion and dispersion of solids are reported. The surface of the neutralized tailing are quite stable after initial drying, probably through cementing with gypsum. This crusting provides an erosion resistant surface during the summer unless repeatedly wet and dried by precipitation. This cycle breaks down the crust, making it more susceptible to local erosion. A similar process is noted during spring breakup; the effects of freezing and thawing and leaching of the surface leaves an unstable, friable surface subject to wind erosion.

III.4.0 SURFACE FROST ACTION

Frost action is of concern in tailings impoundments as it can:

a) result in mass transport of pore water and tailings solids to the surface of the tailings and degrade cover integrity;

b) adversely affect the stability of water retaining embankments; and,

c) adversely affect drainage systems.

III.4.1 THEORY OF FROST ACTION

Frost action requires 3 essential elements.

a) freezing temperatures
b) a frost susceptible soil
c) a free supply of water

The Saskatchewan uranium producing areas are in northern latitudes where severe winters are common. The conditions are sufficiently severe for discontinuous permafrost to occur which becomes more prevalent in the more northerly latitudes. The Ontario uranium producing areas are more moderate in climate, however freezing conditions are common. The approximate freezing indices are 3200, 2700 and 1100 degree days Celsius at Uranium City, Key Lake and Elliot Lake, respectively.

The tailings are predominantly composed of silt and very fine sand sized particles and are frost susceptible. The uranium producing areas are all north of the sedimentary basin. Thus, fine grained clay soil is not available for embankment construction and frost susceptible silt or silty sand is the least permeable soil available. Therefore both the tailings themselves and the borrow soil available for embankment and cover construction are generally
frost susceptible.

Most producing mines use a wet tailings deposition and high water table conditions are common in the uranium producing areas. In the remaining mines, tailings are only partially dewatered, and drained. Therefore free water is available within the tailings mass in sufficient quantity for ice accumulation and frost heave to occur.

The conditions required for frost heave, namely frost susceptible soil, freezing temperatures and free water, are present within the tailings mass and within associated embankments and control structures. Long term integrity of the tailings mass, covers and structures must consider frost effects.

Frost action analysis requires a predictive model for both the thermal regime and for the heaving process. Several thermal modelling techniques ranging in sophistication are available. The simple models consider heat flow through the upper frozen zone (freezing soil) and the latent heat of freezing at the freezing front (Nixon and McRoberts, 1973). Models of increasing sophistication account for varying initial ground temperature,

determined in frost cell tests and applied to conventional seepage flow models.

Thaw consolidation models are available to predict the flow of water from a thawing soil (Morgenstern and Nixon, 1971). The soil properties required can be determined from frost cell tests and the thermal regime from thermal modelling. The theory was developed to predict the settlement of a thawing ice rich soil but can readily be adapted to characterize the flow of water from a thawing soil.

Research work has been carried out on thermal gradient induced water flow, primarily as a component of frost heave research. Limited information that can be applied to contaminant transfer is available. It is unlikely that this flow mechanism is significant in relationship to heaving induced flow.

In summary,

1) thermal modelling is quite well developed and is available for application

2) frost heave theory is available but not yet a proven technology

3) thaw consolidation theory is available to estimate the flow of water from thawing soil

4) laboratory testing procedures have been developed to provide soil properties for all the above tests

5) gradient induced flow theory is available but not likely developed to a practical stage.

III.4.2 SURFACE FROST PENETRATION

Surface frost action on both the tailings mass and on the earth control structures can result in transport of tailings pore fluid to the surface and potential release.

The potential flow mechanisms are:

1) ice segregational frost heave
2) induced excess pore water pressures, and
3) thermodynamic induced flow

Ice segregational heave is the formation of ice lenses in the soil as the freezing front progresses. The freezing front generates negative pore water pressures which draws water to the front where it freezes into ice lenses. When the ice melts, excess pore pressures are induced. As thawing occurs from the top, the excess pore water cannot flow downward because of the underlying yet frozen zone and upward water flow occurs. Succeeding freeze/thaw cycles therefore have the potential of "pumping" water to the surface.

The second mechanism is the direct generation of excess pore pressures by freezing. Under high rate of freezing conditions and in coarser grained

specific heat of the frozen and unfrozen soil and varying surface heat transfer conditions. Closed form analytical solutions predict the depth of frost penetration but have limited application in determining temperature gradients.

Numerical models are available (Hwang; et al, 1972, Kent and Hwang; 1979) which can rationally account for heat exchange at the ground surface taking into account such factors as air temperature, wind, snow cover, solar radiation, evapotranspiration, shadows and surface albedo and emissivity. They provide the temperature regime throughout the frozen and unfrozen soil, temperature gradients and the rate of change of temperature gradients.

The soil data required for a thermal model consists of the unit weight and moisture content of the soil from which the volumetric latent heat, frozen and unfrozen thermal conductivity and frozen and unfrozen specific heat can be determined from known relationships developed for typical soils. For tailings which may have atypical petrology, specific relationships may have to be developed. Consideration of the effect of highly mineralized pore water on thermal properties is also required. Its effect on properties such as freezing point suppression may be significant.

The frost heave mechanism has been the object of considerable study. The mechanisms of heave are well understood, however, predictive heave models are in their infancy.

When soil freezes, a change in pore water pressure occurs. Depending upon the pore size distribution and the rate of freezing, positive or negative (suction) pressures are developed. Suction pressures, which result in ice segregational heave are more common.

The negative pore pressure draws water to the freezing front where it freezes to form ice lenses. The ice concentration results in frost heave. The suction pressure is a result of the interfacial tension between the ice and water within a soil pore. It is analagous to the suction pressures developed in an unsaturated soil by "capillary action" or the interfacial tension between the air and water phases. The magnitude of the negative pressure increases as the pore size decreases (ie. greater in fine grained soil).

Konrad and Morgenstern (1984) have presented a theory and method for frost heave estimation. Such predictions have been elusive, and the Konrad work appears to hold the most promise. Their theory considers the effects of the rate of freezing, applied surcharge and the suction developed at the freezing front. The rate of freezing and ultimately the amount of segregated ice likely to form can be predicted from a limited series of laboratory frost cell tests. The theory has been verified in the laboratory on field scale chilled pipeline tests.

In coarser grained soil under rapid freezing conditions, positive pore pressures can be developed and water expelled from the freezing front. The expelled water can force itself to the surface through weak spots in the frozen crust. There is little theoretical work on this mechanism in the literature. However, the magnitude of pore pressure generation can be

(fine sand) soil, water is expelled from the freezing front by the phase volume change water. The expelled water increases the pore water pressure in the underlying unfrozen soil. The water is prevented from flowing to the surface by a surficial frozen crust.

If there is a weak spot in the crust, the water can flow to surface, freeze, or create a raised, ice cored mound. On a larger scale, this is the geomorphology of pingos in the western arctic. Upon subsequent thaw, water can be released to the surface.

The third mechanism is flow resulting from thermal gradients within the soil. This mechanism is the direct generation of pore water pressure gradients by a temperature gradient. Its generation does not rely upon transient freezing. The free energy of ice and cool water is less than warm water. In an attempt to maintain equilibrium energy conditions, flow in the soil water system will occur from warm to cool areas. The flow can occur both through thawed soil and through the unfrozen water present in fine grained soil at sub freezing temperatures. This mechanism has been studied as a component of frost heave research and is of secondary importance to ice segregational mechanisms.

An example of frost action is the frost features present in the Gunnar Mine tailing surface. Although not associated with structures, these features show the potential for frost activity. Here frost action has developed locally raised, palsa like, structures a few metres in width on the tailings surface. The raised structures are more exposed in winter because of elevation and loss of vegetation and/or snow cover which tends to increase frost activity and continued growth. Because the structures are raised, the soil is subjected to increased erosion and transport. Associated with the mounds is an ice core which can be released to surface upon thaw.

Frost action can also result in deterioration of water retaining earth embankments. In uranium producing areas, clay borrow is generally limited and frost susceptible silt and sandy silt must be utilized for embankment construction.

As the freezing front progresses from the dam crest and exposed sideslopes, frost heaving occurs with free water from the reservoir drawn to the freezing front where ice lenses form. Upon thaw, the water from the ice lenses generates excess pore water pressure. Sufficient excess pore water pressure can be generated to result in local slumping of the embankment. If unchecked, the process is continuous with additional slumping occurring each spring. Solymarr and Nunn (1983) reported damage of this type on the Waterloo Dam near Uranium City, Saskatchewan as well as two other dams where design measures were undertaken to prevent frost damage. In the Waterloo Dam sufficient excess pore pressures were developed not only to result in slumping but to cause the silt to liquefy and flow to surface through the tension cracks created during slumping. Similar mechanisms may occur in Northern Saskatchewan and to a reduced extent at Elliot Lake where winter conditions are less severe.

Frost indicators installed into frost caps over glacial till cores on the Denison tailings dams at Elliot Lake indicate frost penetration of 1.3 to 1.6 m into the cap.

The technology available to control frost damage consists of:

a) limiting frost penetration by insulation

b) limiting frost penetration in the silt by constructing a non frost susceptible granular fill in conjunction with a geomembrane for water tightness

c) using bentonite conditioned fill to reduce the silt permeability and thus potential frost heaving.

Each of these measures can be utilized on existing embankments.

Another method is to artificially create a permafrost core in the frost susceptible zone. This is an untried method. One concept consists of surficial insulation plus passive heat radiators "thermo-piles" or such as used for pile stabilization on the Aleyeska oil pipeline or similar devices. This may not be acceptable because of the maintenance required. Other passive techniques can also be evaluated.

Drains

Drains required for structure integrity or for control of contaminant migration are subject to blockage by freezing. Slow fluid discharges into freezing conditions results in icing and temporary blocking of the drain. Shallow buried sections of the drain may fall within the depth of seasonal frost and may also be blocked. There is also a potential for massive frost heaving at the drain elevation which could result in structural distress to rigid drainage pipe. Under extreme winter events, a depth of frost from 3.5 to 4 m is possible in northern Saskatchewan.

Insulation is the available technique for preventing frost penetration to buried drains. Design guidelines for the amount and placement of insulation are available.

Icing at the discharge is more difficult to control. Extension of the drain and discharge into a pool of water is one alternative. Heating is a second alternative, however, this requires maintenance and energy.

III.5.0 SURFACE BIOLOGICAL ACTIVITY

III.5.1 ROOT ACTION

In the absence of a vegetative cover, precipitation reaching a tailings ponds surface would be removed by evaporation, surface runoff, and percolation.

On land covered with vegetation, evapotranspiration is the dominant mechanism governing soil water losses. Water loss from a soil to the atmosphere may occur by evaporation at the soil surface, and by transpiration of near surface soil water absorbed by plant roots and transported to the plant leaves. The combined loss resulting from these two processes is termed evapotranspiration. Potential evapotranspiration is determined by several physical characteristics of climate and soils; actual evapotranspiration is determined by the interaction of these physical factors with the type and degree of vegetation cover.

Numerous methods have been developed to predict both potential and actual evapotranspiration (Hanks and Ashcroft, 1980). Through the use of one or more of the methods, it is possible to predict the proportion of precipitation transpired back to the atmosphere by vegetation, the proportion remaining in the soil, and the proportion available for seepage from the soil.

To aid in designing the cover system for the tailing disposal facility at a proposed mine in the northern section of Wisconsin, U.S.A., a detailed study was undertaken to determine the potential water balance of the tailings disposal facility area assuming a vegetative cover was established on the tailings pond reclamation cover. The Thornthwaite and Mather method (Thornthwaite and Mather, 1957) was selected as the most suitable method for determining evapotranspiration potential of the proposed waste disposal facility area.

A water balance model was designed and used to analyze a number of climatic conditions and three cover systems on the tailings ponds. The three cover systems were:

System 1: 0.91 m (3 foot) glacial till underlain by a 0.15 m (6 inches) bentonite modified soil seal layer.

System 2: 0.91 m (3 foot) glacial till underlain by a 0.20 m (8 inches) coarse drain, in turn, underlain by a 0.15 m (6 inches) bentonite modified soil seal layer.

System 3: 0.71 m (28 inches) glacial till underlain by a 0.20 m (8 inches) coarse drain, in turn underlain by a 0.15 m (6 inches) bentonite modified soil seal layer.

Three climatic conditions, based on precipitation data obtained over a 42-year period at a nearby Federal weather station, were used in the analysis.

Average annual precipitation for the 42-year period, 763 mm (30 inches), was used to represent the first climatic condition. During this period of

record, precipitation totaled 448 mm (18 inches) in the driest year, whereas the wettest year on record was 1,014 mm (40 inches). The driest and wettest years were used to represent the second and third climatic conditions.

Actual evapotranspiration will, in part, be determined by the type and degree of vegetative cover present (Pritchett, 1979; Hanks and Ashcroft, 1980). For the purpose of the analysis, it was assumed that the vegetative cover was composed only of black spruce (Picea mariana) and that the soil surface was completely covered by the forest canopy. Black spruce was selected for two reasons (1) it is a species native to the proposed mine area, and (2) the role of black spruce in determining the water balance of various sites had previously been investigated (Verry, 1976; Verry and Timmons, 1982).

The water balance modelling and analysis used monthly data for each of the following parameters: potential evapotranspiration, precipitation in an average, dry and wet year, and surface runoff. Moisture available for infiltration, initial soil moisture, total available soil moisture, actual evapotranspiration, remaining available moisture and percolation were determined through the model.

With System 1 as the tailings cover design, the water balance model predicted wide fluctuations in surface runoff, available soil water and percolation. For the wettest year, the 0.91 m (3 foot) glacial till overburden would be completely saturated for a 4-month period beginning in August. The degree of overburden saturation would result in a greatly increased level of surface runoff as well as the creation of a soil environment detrimental to vegetative growth. In a year of average precipitation, an excessively dry overburden (available soil moisture below the plant wilt point) would occur during August. During the dry year, an excessively dry overburden would occur for 5 months, June to October. Total percolation through the cover to the underlying tailings was predicted at 15 mm (0.6 inches) in the dry year, 40 mm (1.6 inches) for an average year, and 113 mm (4.5 inches) in the wettest year. The percolation in each case would normally occur in the March-May period when infiltration from snowmelt and rainfall would exceed the level of evapotranspiration occurring at that time.

Cover designs for Systems 2 and 3 differed from that used in System 1 by the inclusion of a 0.20 m (8 inch) thick coarse drain layer between the overburden layer and the bentonite modified soil seal. System 2 differed from System 3 only in the depth of glacial till overburden placed above the coarse drain and bentonite modified soil layer, 0.91 m (36 inches) versus 0.71 m (28 inches), respectively. Inclusion of a coarse drain layer prevented saturation of the glacial till overburden by admitting free drainage from the overburden at all times. In the wettest year, no adverse soil water conditions would occur. Potential difficulties with Systems 2 and 3 could occur during dry periods. The water balance calculations indicate that in an average year, dry soil moisture conditions would occur in August and September, whereas in the driest year, such conditions would exist for 5 months. Such patterns of soil moisture storage, availability and drainage in System 2 and 3 would actually reflect naturally occurring conditions on similar soils in undisturbed areas of the Project site.

Condensed data from the water balance model are presented for each of the alternative design systems and for the three climatic types in Table III.5-1. System 1 provides the greatest evapotranspiration and System 3 the least. The difference in evapotranspiration between the two systems is caused by the lesser soil water storage capacity of the shallower overburden layer in System 3, and hence, a lower amount of water is available for evapotranspiration.

A comparison of percolation in System 1 with the vertical percolation in Systems 2 and 3 demonstrates the efficiency of the drainage layer in the latter systems in reducing percolation to the tailings. The horizontal percolation (lateral drainage) will exit those two systems through the drainage layer and would become surface runoff outside the perimeter of the tailings ponds. Based on the results of the water balance study, System 2 would be the preferred cover design if the costs of acquiring and placing the additional overburden are disregarded. With System 2, approximately 5 percent more soil moisture is available for plant growth than in System 3 during a dry year and it provides a deeper medium for plant root growth, thus decreasing the probability of plant roots entering the drainage layer or the bentonite modified soil layer during periods of depleted soil moisture.

The results of the aforementioned vegetation-overburden-water balance study have important implications in the design of tailings disposal areas reclamation programs. Often the stated objective of a reclamation-revegetation program is that the tailings surface will be revegetated with the long-term objective of encouraging the development of a forest-type vegetative cover similar to that indigeneous to undisturbed land adjacent to the tailings disposal area.

Earlier concepts that direct seeding of an herbaceous cover on a tailings site would allow classical plant succession processes to occur, which would result in the development of an indigenous forest-type vegetative cover, are being revised as more long term data and observations become available (see section III.2.7). It is certainly true that the proposed practice of covering a tailings site with local glacial till materials included the underlying assumption that such material would be more inducive to the development of an locally indigenous forest-cover. Whether the above assumption is correct or incorrect is still not known given the relatively short time period for which experience is available from tailings reclamation programs in Canada today. Experience from agricultural and construction fields would indicate that the 'question' must remain open for the time being. It is known that where a coarse material overlies a finer material, hard pans may form due to the impedance of the flow of dissolved or suspended solids (e.g., Lutz, 1952; Ingles and Metcalf, 1973). Placement of soil materials in the manner described above may result in the occurrence of perched water tables in the coarser horizon (Ingles and Metcalf, 1973); unless of course the final surface grade of the tailings is sufficient to permit free lateral run-off.

Where one contrasting material is placed over another (ie., overburden directly over tailings) or where a third layer (i.e., a sealant layer) is interposed between two other layers as in systems 1, 2 and 3 discussed

TABLE III.5-1

WATER BALANCE SUMMARY FOR THREE ALTERNATIVE TAILINGS CAP CONSTRUCTION SYSTEMS

SEASONAL VARIATION	PRECIPITATION (mm)	RUNOFF (mm)	EVAPOTRANSPIRATION* (mm)	VERTICAL PERCOLATION (mm)	HORIZONTAL PERCOLATION (mm)
SYSTEM 1					
Normal Year	762.8	128.8	594.1	39.6	
Driest Year	447.8	115.8	360.4	39.6	
Wettest Year	1014.0	306.3	561.3	112.8	
SYSTEM 2					
Normal Year	762.8	128.8	564.6	16.0	53.1
Driest Year	447.8	115.8	322.1	16.0	62.0
Wettest Year	1014.0	118.6	560.6	18.8	316.0
SYSTEM 3					
Normal Year	762.8	128.8	548.4	16.3	67.8
Driest Year	447.8	115.8	305.8	16.0	62.0
Wettest Year	1014.0	118.6	560.6	18.8	316.0

Evapotranspiration includes plant interception estimated as 20 to 30 percent of total precipitation.

earlier, it is clear that a plant rooting environment will be created which is atypical of that of the area surrounding a tailings disposal facility. The atypical rooting environment will undoubtedly have an important influence on the ultimate development of a forest-type vegetation cover; but in most respects will not have an influence, at least in the short term (<10 years) on an herbaceous cover initially established to control surface erosion by wind and water.

The influence of the rooting environment on potential long term development of a forest-type cover arises from the fact that forest tree root systems are not universally equal in form, and tree species vary in their adaptability to physical and chemical soil conditions.

Although the habit, or form, of a root system is influenced by local site conditions, it tends to be under genetic control. Most tree root systems can be conveniently grouped into: 1) tap, 2) heart, or 3) sinker (flat) forms (Pritchett, 1979). Tap-rooted trees are characterized by a strong downward growing main root, which may branch to some degree, as in most Carya (Hickory), Juglans (Walnut), Quercus (Oak), Pinus (Pines) and Abies (Firs) species. Numerous strong roots radiating diagonally from the base of the tree without a strong tap root is characteristic of heart-root trees, as generally found in Latrix (Larch), Betula (Beech), Carpinus (Ironwood) and Tilia (Basswood). The sinker root habit is dominated by strong laterals from which vertical sinkers grow straight down as in Populus (Poplar), Fraxinus (Ash) and some Picea (Spruce).

Some general characteristics of root forms in relation to certain soil depths and general horizon properties can be considered. For species with inherent shallow or flat root systems, deep permeable soils with little or no restriction to rooting within the profile offer no advantage. Conversely, shallow soils or those in which fluctuating water-tables may restrict deeper roots can be successfully exploited by flatrooted species. For example, black spruce may be commonly found over a wide range of soils - thin soils over bedrock, organic soils with a fluctuating water-table within 30 cm of the surface and on deep well-drained soils of various textures. In a wide range of conditions the roots of this species were found mainly (80 percent) in the upper 60 cm and in many stands were not found below 45 cm (Schulz, 1969). This same species is also adapted to the increase in thickness or organic peat in which it may be growing by the production of adventitious roots from stem and branches.

In soils where root penetration is restricted by stones and shallowness over a fissured bedrock, those species in which the root form consists of laterals, some oblique roots and sinkers have an advantage over other root forms. This is true for eastern white pine, in which the root system is capable of exploiting both lateral and vertical fissures in a fractured bedrock. Occupation of soil by roots is neither uniform nor static and this applies to individual plants as well as communities of plants. As plants develop, both seasonally and over their lifespan, there is continual change in the root system. Some roots extend into 'new' soil, while mortality causes a reduction elsewhere in the root system. Also the growth and development of roots is controlled not only by soil environment factors such as temperature, moisture and fertility, but also by the condition and

physiological status of the above-ground portion of the plant on which the root system is dependent for photosynthetic and growth substances.

The occurrence of soil layers which impede the free drainage of water is greatly increased during the spreading of overburden and topsoil in land reclamation schemes. The existence of such layers is caused by compaction due to soil spreading and soil grading vehicles with severe detrimental effects on subsequent revegetation programs (Richardson, 1977, Watson and Luxmoore, 1979). Unless remedial measures are undertaken prior to seeding or planting, the soils remain poorly drained, often exhibit surface water ponding and provide an unsuitable medium for herbaceous and tree growth.

Root growing space, measured as depth of soil above some root restricting layer or water table, has been used as an indicator of site productivity more than most physical factors because it can be measured with some accuracy in the field and it is an integrator of many other factors important to tree growth. Except in very shallow soils, depth has little direct influence on tree growth. It has considerable indirect effect as it influences available water storage capacity, nutrient availability, and aeration (Pritchett, 1979). When soil depth is such that it defines the volume of growing space for tree roots above some restricting layer, such as claypan, siltpan, bedrock, or other horizon of low permeability, depth measurements can be used with some precision to predict growth patterns. Growth normally follows a trend that can be expressed as a reciprocal function of soil depth with general decline in growth found on soils with less than 25 cm of effective depth.

The absolute and effective depths of a soil are not necessarily the same, because a high water table, toxic substances, or an impervious layer may completely restrict root penetration in a soil that would otherwise permit deep rooting. Some difficulty in using soil depth to estimate productivity also may be encountered where drought erosion, or pore drainage are products of surface soil thickness or depth of soil above some restricting layer. In each of the instances, there may be soil fertility interactions important to tree growth that cannot be determined from depth measurement alone. Optimum soil depths quoted by Pritchett (1979) range from 50 to 100 cm depending upon species and region.

The authors have not been able to locate any definitive scientific articles on the relationship between tree root growth and overburden depth in land reclamation programs in the United States and Canada.

Studies of "topsoiling" or emplacing thin covers of available sediments over spoils have been conducted (e.g., Grandt, 1978, Sandoval and Gould, 1978). In many areas, the spoils constitute a growth medium for crop roots and a thin topsoil layer suffices to form a workable seedbed for getting crops established. In areas with highly acid or sodic type soils, that approach is unacceptable because the total root zone is confined to the shallow cover layer above the spoil (e.g., Schuman and Power, 1981). In such instances, overburden depths of 61 to 91 cm and a superimposed topsoil layer of 30 to 61 cm may be required, the former acting as a barrier to upward migration of acid or sodic salts. Typically, most of these studies have been conducted in the central or western States and mining areas. In the above studies, the effect of the soil environment on root growth has

been assessed through patterns of shoot growth only and not by direct measurement of changes in root morphology or physiology.

Observations of the type reported by Hall (in Anon, 1980) are rare. In a study conducted on disturbed coal mining land covered with overburden (depth not stated) it was found, after three growing seasons, that European alder (Alnus glutinosa) was capable of sending a sinker root system down through the compacted, high-clay content soil to a depth of approximately 91 cm, whilst the roots of the hybrid poplar (Populus alba x P. grandidentata) were confined to wide horizontal spread in the upper 15 cm of soil.

Concomitant with the placement of one or more layered capping systems on a tailings surface is the question of soil compaction and its relationship to vegetative cover development.

Soil compaction can be considered from the viewpoints of the civil engineer and of the agronomist. Traditionally, the civil engineer has considered the soil as a construction material, as a foundation or as a tractable surface. The process of soil compaction is an engineering technique to accomplish the desirable goal of forming the densest and tightest possible soil condition. To the agronomist, interested in the soil as a medium for plant growth, soil compaction is a scourge, an undesirable consequence of mechanization which must be avoided lest it result in the soil becoming unfit for crop production (Hillel, 1980). In the agronomic context, soils or soil layers are considered to be compacted when the total porosity, and particularly the air-filled porosity, are so low as to restrict aeration, as well as when the soil is so tight, and its pores so small, as to impede root penetration and drainage. Soil and soil layers may become compacted naturally as a consequence of their textural composition, moisture regime, or the manner in which they were formed.

The effect of pore size has been clearly demonstrated by Wiersun (1957), who grew seedlings of a number of plant species above sintered glass discs with different pore sizes. Only those roots with diameters smaller than the pore diameter passed through the discs. Seedlings were then grown in sand culture placed in glass tubes of varying diameter to give a range in rigidity of the matrix. The smaller tubes that would not permit the expansion of the pores prevented root passage, but roots easily passed through the sand in the large tubes. Thus, it seems clear that roots cannot accommodate to an opening smaller than that of the root tip. It would follow that roots will not penetrate compacted soil if their diameter is large in comparison with pore size and the structure is relatively rigid, as in the case of a traffic pan at intermediate or high moisture tension. The same would be true for indurated pans.

The case for the absence of root systems in sandy layers with a low silt and humus content, will often be the mechanical obstruction of the penetration and secondary growth of roots. From examined natural profiles and from artificially built profiles, it appears that rooting is prevented by means of mechanical resistance when the pore volume is less than 40% of the total soil volume.

If soils are sufficiently compacted, the roots will not enter at all. This was demonstrated by Veihmeyer and Hendrickson (1948) who found that there was a maximum density for root growth. When a soil sample has a density greater than this maximum value, roots would not enter. This was not the same for all soils that they studied but varied from about 1.75 g/cm^3 for the sands down to about 1.46 g/cm^3 for the clays. They also found that the roots penetrated saturated soils of low density even through the air had been expelled; hence, it appears that aeration was not the limiting factor. This suggests that the dense soil with small pores may have been the factor that limited root penetration.

One of the major difficulties in the field examination of soils or in experiments is to separate the effects of a simple mechanical barrier from related changes in soil moisture status and aeration. It is not always appreciated that porous, coarse-textured sands and fine gravels constitute a very rigid soil system and can create as much mechanical impedance to a root as a compacted or cemented soil horizon such as a fragipan. Two examples can present extremes in aeration and soil moisture supply; a coarse-textured soil with low water-holding capacity but porous and well-aerated and a fragipan with impeded drainage and seasonally reduced aeration. Reduced growth of planted Douglas-fir seedlings has been attributed to compaction following tractor logging. Youngberg (1959) found that growth of seedlings was significantly greater on a cutover where soil bulk densities to 30 cm were between 0.87 and 0.98 g/cm^3 than growth in roadways or berms where soil bulk densities to the same depth ranged from 0.88 to 1.73 g/cm^3. Red pine seedlings showed reduced primary root development in a sandy loam at bulk density of 1.45 g/cm^3 compared with growth in the same soil material with a bulk density of 1.02 g/cm^3. (Armson and Williams, 1960).

In many soils, although compact or impermeable layers may restrict general root development, there are often places where a vertical root manages to grow through and, if more favourable soil zones exist under the impermeable layer, the root will proliferate in this region. Once the original root channel has been created, succeeding generations of roots may make use of it.

For a given soil, bulk density can be correlated with gas diffusion rate, soil "strength", and water transmission characteristics - but not amongst different soils. As yet, no generally accepted means of characterizing or expressing soil strength has been devised. Strength varies tremendously with degree of compaction and even more with moisture (Shaw et al, 1942; Gill, 1959; Taylor and Gardner, 1963). Not only moisture level but also antecedent moisture is important.

Thus, the often implied concept of a critical bulk density for root growth is not strictly acceptable. A density that would deny root penetration at one moisture content may offer little impedance at another. To be useful in predicting or interpreting root growth, bulk density must be related to soil moisture and texture. However, until an acceptable method of measuring soil strength is found, bulk density will remain a most useful criterion of mechanical impedance. The needle penetrometer provides a reliable means of evaluating soil resistance to root penetration, even though it is influenced by factors that only indirectly affect strength.

III.5.2 BURROWING INTRUSION

Beedlow and Hartley (1984) and Beedlow and Parker (1985), based on USA observations have concluded that burrowing activities by biota may result in a slow but long term transport and release of tailings. From the known fauna of northern Saskatchewan and Elliot Lake regions, (TMI.2.3) neither burrowing or nesting activity by rodents and birds would be considered a major cause of instability in and around tailings impoundment sites. Such activity could cause minor exposure of the tailings or local erosion. There appears to be no information available which quantifies the potential effect of burrowing and nesting activities on either a minor or major scale. To what extent these activities may lead to piping development is unknown but it is considered to be minor. It is concluded that burrowing intrusion would be considered a significant cause of instability only where the tailings are highly radioactive and toxic and where small releases are in themselves significant.

Human activity on tailings dams whether from recreational activity (e.g., motorcycles) or from reuse of tailings in construction programs would be considered to have far greater erosion effects but detailed consideration of these non-natural activities is outside the scope of this report.

III.6.0 SUBSURFACE LEACHING AND SEEPAGE

III.6.1 INTRODUCTION

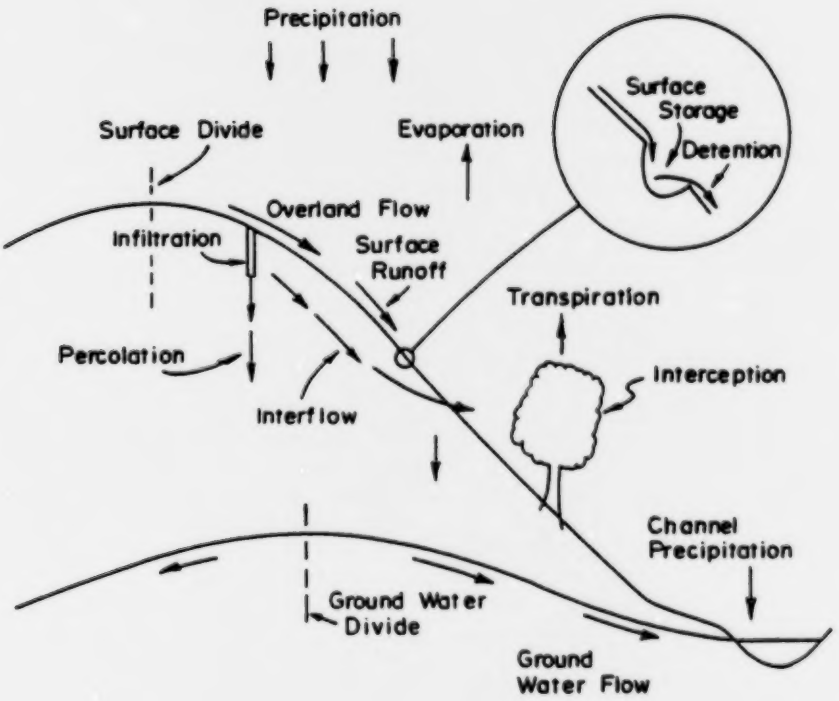
Infiltration is defined as the passage of water through the surface of the soil, via pores or small openings, into the soil profile. It is one of the most important components of the hydrologic cycle. The ability of water to move through a soil profile has a direct effect on many of the other components of the hydrologic cycle. The main components of the hydrologic cycle are climatic conditions, evapotranspiration, interception, detention storage, infiltration, overland flow, subsurface flow, surface storage, deep percolation, ground water flow and streamflow. Infiltration directly affects subsurface flow, deep percolation, (see Figure III.6-1) ground water flow and surface runoff. Available water in the soil profile has a major influence on the vegetative growth and does therefore, indirectly influence evapotranspiration, interception, and surface storage.

In the long term the infiltration into and seepage out of the impoundment must be in balance. Net seepage is therefore equal to net infiltration and can be determined from infiltration estimates.

Historically tailings have usually been deposited by a wet deposition method. Water is normally ponded at the surface of the impoundment and the bulk of the tailings profile is saturated. Generally, localized mounding of near surface ground water systems will occur. Containment slopes might have been constructed from coarse tailings material, waste rock, or locally available earth and rock materials. In some cases a subsurface drainage system might have been installed to protect the containment slopes or to prevent contaminant seepage migration into the underlying foundation materials.

Following active deposition the impoundments are reclaimed possibly by placing an earthen cap over all the tailings surfaces. Prior to reclamation, partial dewatering of the tailings might be necessary. If relatively good underdrainage conditions exist, then in the long term the bulk of the tailings will become unsaturated and localized mounding of ground water will dissipate. Such good underdrainage conditions do not exist at most Canadian uranium tailings impoundments. The main sources of recharge to the tailings will be due to infiltration of rainfall and upstream runoff, or from inflows associated with ground water rises during wet periods.

It can be seen that infiltration will play a major role in the long term success of a natural earth cover. The development of a vegetative cover will depend on the availability of water in the cover (but not an excess) and the ability of roots to penetrate through the cover. Therefore, some infiltration is necessary to maintain vegetative growth. Also, if infiltration is low, surface runoff will be high and will result in accelerated cover losses due to erosion. If infiltration rates are high the potential for mounding and subsurface contaminant migration will be increased. If mounding reaches the root zone or the surface it may result in loss of vegetation cover.



THE HYDROLOGIC CYCLE (SOURCE: HAAN AND BARFIELD, 1978)

FIGURE III.6-1

The infiltration process must therefore be understood and infiltration estimates made. Infiltration and flow movement through the cover materials and surficial tailings will generally occur as unsaturated flow. Seepage and contamination migration out of the tailings could occur either as saturated or unsaturated flow.

III.6.2 INFILTRATION THEORY

In 1856, Henry Darcy developed the basic relationship for describing the flow of water through a homogeneous soil. In Darcy's experiments he used sand as the porous media and concluded that the rate of flow was proportional to the hydraulic gradient. Darcy's law may be written as:

$$q = -K\partial\Phi/\partial s \qquad (6.1)$$

where q is the flux, or volume of water moving through the soil in the s-direction per unit area per unit time; and $\partial\phi/\partial s$ is the hydraulic gradient in the s-direction. The hydraulic conductivity K is dependent on the properties of the fluid and the porous media. Darcy's experiments were limited to one-dimensional flow, but his results have been found to be applicable to three-dimensional flow.

III.6.3 EMPIRICAL INFILTRATION MODELS

This section will present a review of infiltration models which have little or no physical meaning. Although several of these models can be calibrated using physical properties of the watershed, the properties used have little physical significance in quantifying soil water movement.

Kostiakov Equation

In 1932, Kostiakov proposed one of the simplest infiltration equations:

$$f = K_k\, t^{-a} \qquad (6.2)$$

where f is the infiltration capacity, t is the time from the start of infiltration, and a and K_k are best fit parameters determined experimentally. The parameters K_k and a have no physical significance.

Israelson and Hansen (1967) developed a modified version of the Kostiakov equation, which is very similar to Horton's Equation, by adding an equilibrium infiltration capacity term. A further modification was made by Smith (1972) who developed a form of the Kostiakov equation for two-stage infiltration events. The Smith form of the equation is:

$$f = f_c + A(t-t_o)^{-b} \qquad (6.3)$$

where A, b, and t_o are empirical parameters which are determined from experimental results. In studies with six soils, Smith found that over 99 percent of the change in infiltration capacity with time was described by this equation. Like the Horton equation, the goodness of fit could be attributed to the fact that the equation has three parameters.

Horton Equation

One of the most widely used infiltration models is the three parameter equation developed by Horton (1939):

$$f = f_c + (f_o - f_c)\, e^{-\beta t} \qquad (6.4)$$

where f is the infiltration-capacity at time t, f_o is the infiltration-capacity at time zero, f_c is the final constant infiltration-capacity (equilibrium infiltration-capacity), and β is a best fit empirical parameter.

If equation 6.4 is integrated, an equation can be developed for determining the total volume of water infiltrated:

$$F = f_c t + (1/\beta)(f_o - f_c)(1 - e^{-\beta t}) \quad [F=0 \text{ at } t=0] \qquad (6.5)$$

where F is the volume of water infiltrated at time t. Both equations 6.4 and 6.5 are valid for ponded infiltration.

Holtan's Equation

An equation with more physical significance than the equations described previously was developed by Holtan (1961) and Holtan et al. (1967). The Holtan equation describes infiltration as a function of the remaining available storage in the root zone. It is expressed as:

$$f = f_c + a(S_t - F)^b \qquad (6.6)$$

where S_t is the storage potential of the soil in the root zone. The constants "a" and "b" are determined as the intercept and slope, respectively, of a log-log plot of $(f - f_c)$ versus $(S_t - F)$. F is the accumulated infiltration volume and f_c is the steady state infiltration rate. The use of F instead of t as the independent variable offers two main advantages: 1) it relates the infiltration capacity directly to soil moisture, and 2) the difficulty of determining infiltration capacity at any time during the event is not encountered. Both advantages are particularly useful in simulations of intermittent rainfall and runoff events.

In a study by Overton (1964), Holtan's equation (equation 6.6) was modified by giving the exponent "b" a value of 2.0. the modification was made to facilitate ease of integration of the equation. Overton showed that the modified equation could be used without a significant loss of accuracy from the original work of Holtan (1961). This observation is consistent with the comments made earlier about the Skaggs et al. (1969) study. In that study the average value of "b" was 3.0 and the median value was 2.3. The major disadvantage of the Holtan equation is the difficulty in selecting a control depth for S_t. Holtan et al. (1975) suggest using the depth of the "A" horizon in agricultural soils, while in an earlier publication, Holtan and Creitz (1967) recommend using either the depth of the plow layer or the depth to the impeding layer.

Huggins and Monke (1967) modified Holtan's equation to obtain an equation, with more physical meaning, which could be applied during intermittent ponded infiltration. The Huggins and Monke equation can be written:

$$f = f_c + a((S_t - F)/TP)^n \qquad (6.7)$$

where TP is the product of the total porosity and the control depth.

Huggins and Monke (1967) proposed that the following procedure be used to determine drainage from the control zone:

(1) when the moisture content in the control zone is less than field capacity, the drainage rate is taken to be zero;

(2) when the soil is saturated, the drainage rate is equal to the infiltration rate; and

(3) when the water content is between field capacity and saturation, the drainage rate can be determined using the equation:

$$\text{Drainage rate} = f_c(1 - P_u/G)^3 \qquad (6.8)$$

where P_u is the unsaturated pore volume at a given time and G is the total porosity minus field capacity.

III.6.4 MEASURED PARAMETER EQUATIONS

Philip's Equation

In 1957, Philip developed an analytic solution to the Richards equation (equation A.6.1.5 in Appendix III.6.1). The solution is in the form of an infinite series and is only valid for ponded infiltration into a homogeneous, semi-infinite soil with a uniform initial moisture content. The equation is a power function series in t :

$$x = \phi t^{1/2} + \chi t + \psi t^{3/4} + \ldots + f_m(\theta) t^{m/2} + \ldots \qquad (6.9)$$

where x is the depth of the wetting front, t is time from the start of ponded infiltration, and the coefficients ϕ, χ, ψ ...., $f_m(\theta)$ are functions of θ, which are the solutions of a series of ordinary differential equations. The terms in equation 6.9 normally converge very rapidly and only the first few terms are needed. The most commonly used form of Philip's equation is a two term truncated equation written in terms of the infiltration rate or the volume of infiltration:

$$f = \frac{St^{1/2}}{2} + A \qquad (6.10)$$

or

$$F = St^{1/2} + At \qquad (6.11)$$

where S is the sorptivity and A is a function of the soil and the initial moisture content. Both S and A can be determined if the soil properties

D(θ) and h(θ) are known. For infiltration over long time periods the parameter A is sometimes interpreted as the saturated hydraulic conductivity (Morel-Seytoux, 1973 and Swartzendruber and Youngs, 1974). Based on the work of Youngs, however, Philip's analytical solution would give a value of one-third the saturated hydraulic conductivity (Youngs, 1968). The validity of the Philip's equation and the goodness of fit of the method has been demonstrated in many research results (Skaggs et al., 1969; Haverkamp et al., 1977; Hanks and Bowers, 1962; Singh and Buapeng, 1977; and Nielson et al., 1961). Application of the method in watershed models has been fairly limited because of the difficulty in estimating S and A.

Green-Ampt Model

In 1911 Green and Ampt developed an analytical solution of the flow equation for infiltration under a constant rainfall. The method is developed directly from Darcy's law and assumes a capillary-tube analogy for flow in a porous soil. The equation can be written as:

$$f = K(H_o + S_w + L)/L \qquad (6.12)$$

where K is the hydraulic conductivity of the transmission zone, H_o is the depth of flow ponded at the surface, S_w is the effective suction at the wetting front, and L is the depth from the surface to the wetting front. The method assumes piston or plug flow and a distinct wetting front between the infiltration zone and the soil at the initial water content (Figure III.6-2). In the equation developed by Green and Ampt, the hydraulic conductivity term K is the fully saturated hydraulic conductivity (Childs, 1969). Bouwer, however, demonstrated that the hydraulic conductivity term should be the resaturated hydraulic conductivity (Bouwer, 1966). The physical significance of the parameters used in the Green and Ampt equation are described by Morel-Seytoux and Khanji (1974).

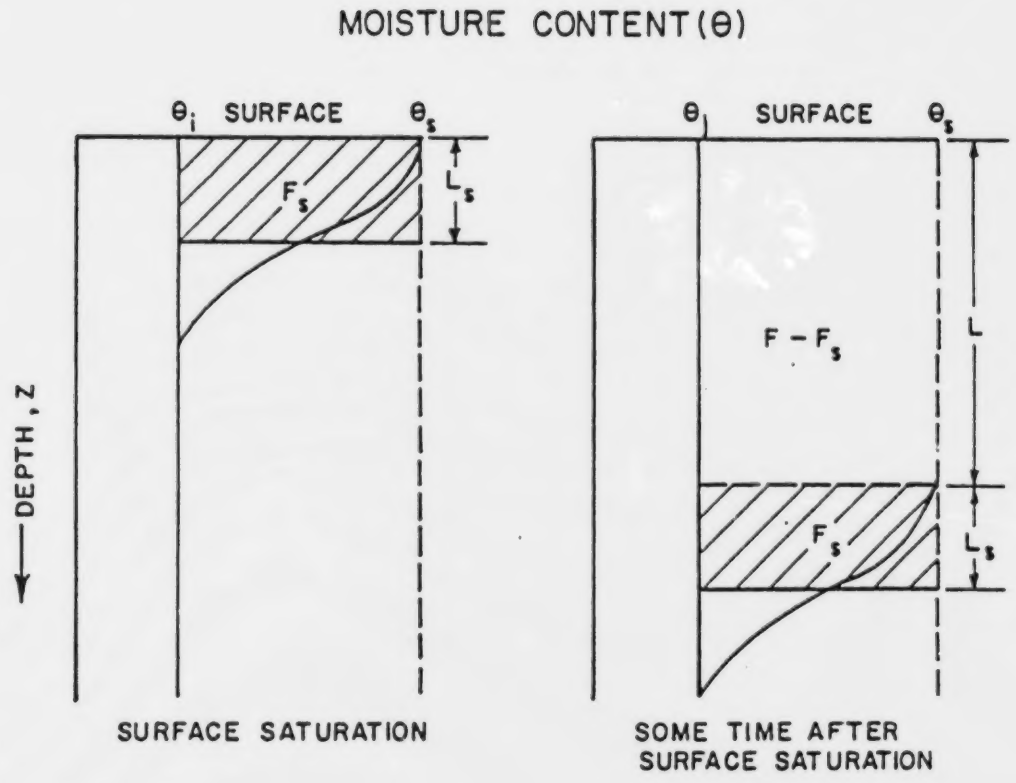
If H_o is assumed to be shallow and can be neglected, equation 6.12 can be integrated to give:

$$Kt = F - S_w n \, Ln \left[1 + \frac{F}{S_w n}\right] \qquad (6.13)$$

where n is the fillable porosity. This form of the equation is generally more convenient to use because it relates the infiltration volume to time. Estimation of the values of the parameters in equations 6.12 and 6.13 has seen considerable attention and a brief review of this research is presented in Appendix A.III.6.3.

Mein and Larson Model

The preceding models of infiltration generally are only applicable to the ponded infiltration. In many situations, application of these models would be very limited. Mein and Larson (1971 and 1973) developed a two-stage form of the Green-Ampt equation. They proposed that infiltration prior to ponding be estimated using the relationship:



GENERALIZED SOIL MOISTURE PROFILES DURING INFILTRATION AT (A) THE MOMENT OF SURFACE SATURATION AND (B) AT A LATER TIME.

FIGURE III.6-2

$$F_s = S_w \Delta\theta / ((I/K) - 1) \quad (6.14)$$

Where I is the rainfall intensity, F_s is the infiltration volume at the time to surface ponding, and $\Delta\theta$ is the initial moisture deficit. The initial moisture deficit is defined as the difference between the saturated moisture content and the initial moisture content. The method is developed for a uniform rainfall intensity, although we will see later how it has been modified for use during a variable rainfall event. Mein and Larson propose that the average capillary suction be used for the suction term, S_w, and that the suction be determined by integrating the suction over the range of the soil moisture.

Chu and Bowers (1977) outline the following procedure for applying the Green-Ampt equation to unsteady rainfall:

(a) Surface not ponded, H = 0

$$f = I \quad (6.15)$$

and

$$F = P - Q \quad (6.16)$$

(b) Surface ponded, H > 0

$$f = K(1 + S_w n/F) = K(1 + S_w n/F_p(T)) \quad (6.17)$$

and

$$F/nS_w - Ln(1 + nS_w/F) = KT/nS_w \quad (6.18)$$

where $T = (t - t_o + t_1)$ and t_o is the time to surface ponding. The time t_1 is the time period for the amount of potential infiltration to equal the rainfall prior to surface ponding. The amount of potential infiltration if F_p is the amount of infiltration prior to surface ponding. F_o is calculated from the relationship:

$$F_s = P(t_o) - Q_o \quad (6.19)$$

Where Q_o is the antecedent runoff. Chu found that this procedure showed good agreement between calculated and measured results with two events contained in the literature, but only fair results with the third event evaluated. The events evaluated are described in Holtan (1961).

Applications of the physically based infiltration models described previously should strictly be limited to homogeneous soils. We have seen that in some cases good estimates are obtained with these models for heterogeneous soils. The goodness of fit, however, should be attributed more to the calibration technique than to the physical basis of the model parameters. Field calibration or regression techniques tend to account for some of the effects of heterogeneous profiles, but do not actually

physically model these effects. Whisler et al. (1972) used a numerical solution of the Richards equation to model a soil in which the resaturated hydraulic conductivity increases linearly with depth. Whisler et al. found that for soils where the permeability increases with depth, the flow initially occurs as piston flow. Bouwer found that the upper layer of the soil will be completely wetted, as with infiltration, into a uniform soil. As the wetting front advances, however, the resaturated hydraulic conductivity increases and the infiltration rate decreases until the wetting front reaches a depth where the resaturated hydraulic conductivity is equal to the infiltration rate. When this depth is reached, the infiltration rate remains constant and the soil is wetted to a moisture content whereby the infiltration rate is equal to the unsaturated hydraulic conductivity of the soil.

Similar results are presented by Hanks and Bowers (1962) for a two-layer system. Using a numerical solution to the Richards equation, Hanks and Bowers found that infiltration through a two-layer system is controlled by the flow through the finer soil. It can be seen that when the finer soil overlays the coarser soil, infiltration through the system is almost identical to infiltration through a uniform profile of the finer soil. Bouwer (1976) developed a procedure for determining infiltration into increasingly permeable soils. The procedure uses the Green-Ampt method and approximates the resaturated average hydraulic conductivity in the wetted zone as:

$$K = \frac{i}{(1/K_1 + 1/K_2 + \ldots + 1/K_i)} \qquad (6.20)$$

where i is the number of layers and K_1, K_2 ... K_3 are the resaturated K-values of each layer in the completely wetted zone. Bouwer also describes procedures for a soil profile which has a linear increase in K with depth. For this situation the average resaturated hydraulic conductivity is estimated by:

$$K = (K_z - K_o)/Ln(K_z/K_o) \qquad (6.21)$$

where K_z is the resaturated hydraulic conductivity at the wetting front and K_o is the resaturated hydraulic conductivity at the surface. A procedure is also presented for estimating z_u, the depth of complete saturation.

$$Y\ LnY = (a/K_o)(h_o + z_u - S_w) \qquad (6.22)$$

where $Y = 1+(a/K_o)z_u$ and "a" is the slope of the linear relationship between the hydraulic conductivity and depth. The equation is solved by trial and error. For other studies on layered profiles, the reader should refer to Miller and Gardner (1962) and Childs and Bybordi (1969). Miller and Gardner compare the results of a laboratory study with results predicted by infiltration equations with the same general form as the Kostiakov equation and Philips truncated equation. They also made an evaluation of the Kirkham and Feng (1949) equation for horizontal flow. Miller and Gardner found that the simple infiltration equations gave poor predictions. Their laboratory observations, however, were similar to the previously described studies with layered systems. Childs and Bybordi compared the results from large layered columns with results predicted

using the Green-Ampt equation and procedures similar to those developed by Bowers (1962). Good agreement was observed between the predicted and observed infiltration rates. Bouwer (1969) also describes a procedure for determining infiltration through a soil profile with a non-uniform initial soil moisture.

In recent years, more attention has been given to the effects of rainfall impact, erosion, vegetative growth, surface topography, and land use activities on the infiltration process. Good accounts of these effects on infiltration are presented by Dixon (1978), Gregory et al. (1979a and 1979b), Gregory et al. (1977), Moore (1979a, 1979b and 1979c), and Hillel (1964). Hillel (1964) and Hillel and Gardner (1969) describe a procedure for determining infiltration through a surface crust. A modification is made to the Green-Ampt equation and the infiltration rate is described by the relationship:

$$f = K_s \left[\frac{h_a}{K_s R_c} \right]^{b/b+1} \qquad (6.23)$$

where h_a is the air-entry value, R_c is the hydraulic resistance per unit area. R_c is approximated as the thickness of the crust divided by the hydraulic conductivity of the crust. The constant "b" is a function of the soil and is dependent on the soil texture and structure. Hillel and Gardner extended this concept into a more rigorous approach which was also applicable during intermittent infiltration. Two equations based on the Green-Ampt approach are established:

$$f = (K^2 R_c^2 (\Delta\theta)^2) + 2K H_f \Delta\theta t)^{1/2} - K R_c \qquad (6.24)$$

where K is the transmission zone conductivity, and H_f is the effective pressure head at the wetting front. This equation is applicable at all but short times in the absence of gravity and at intermediate times in the presence of gravity. For large times a second equation is developed:

$$f = Kt + (H_f - R_c K) Ln(1 + K t / H_f \Delta\theta) \qquad (6.25)$$

The validity of the equations was demonstrated in laboratory studies and it was observed that infiltration into a crusted profile can be described by the approximation that water enters into the subcrust at a nearly constant suction which is a function of the soil-water properties of the crust.

Moore (1979a, 1979b and 1980) describes a numerical procedure for accounting for surface effects on infiltration. Moore developed a complex infiltration model, based on modifications to the Green-Ampt equation, which accounts for surface sealing, erosion, surface storage changes, and soil compaction. The effects of surface sealing were modeled by representing the change in saturated hydraulic conductivity with an exponential decay function:

$$K_s = K_c + (K_i - K_c) e^{-at} \qquad (6.26)$$

where K_c is the final equilibrium conductivity, K_i is the initial conductivity, and "a" is a constant. The equation was found to fit the data of Mannering (1967) and Edwards (1967) for 5 mm surface seals.

Soil splash erosion effects are modeled with a relationship developed by Bubenzer and Jones (1971). The effects of compaction due to erosion and raindrop impact are modeled by the equation:

$$P_b - P_a = a\,t^{1/2} - bM_c \qquad (6.27)$$

where P_b is the porosity of the surface layer before rainfall and P_a is the total porosity after rainfall. M_c is the initial moisture content of the surface layer and "a" and "b" are constants.

Moore (1979b) and Moore and Eigel (1981) describe a procedure which models surface effects during two stage infiltration. The method is based on the GAML model (Mein and Larson, 1971) and is discussed in detail in Appendix III.6.4.

III.6.5 SEEPAGE PREDICTIONS

In the previous section a number of methods for predicting water movement into a soil profile, and through the unsaturated zone, have been presented. Of interest at a reclaimed tailings impoundment will be estimating infiltration into the cover material, through the cover into the tailings, and finally out of the tailings into the underlying foundation materials. Water movement could occur as unsaturated flow, saturated flow, or as a combination of saturated and unsaturated flow.

Water infiltrating through the cover of a reclaimed tailings impoundment may encounter a multilayered system consisting of several different materials. Typically the following profile could occur:

1) a soil cover layer. The thickness of the cover layer will probably be between 0.3-3 metres. Either a layer of rock riprap will be placed over the soil or it will be vegetated. This layer will be unsaturated except following rainfall events. Only in rare cases will the permeability of this cover layer be less than the underlying tailings.

2) a layer of partially saturated tailings which could be 3-50 metres thick. A phreatic surface may occur in this layer. The permeability of the tailings could be higher or lower than the underlying materials.

3) A foundation or liner layer of low permeability material which is normally 0.5-3 metres thick. This layer may consist of compacted local soils or imported clays. However, in some instances the impoundment will be founded on bedrock or a synthetic liner will be used. For these cases, the foundation layer thickness will be very variable and will not usually fall within the 0.5-3 metre range.

McWhorter and Nelson (1979) noted that seepage beneath an uranium tailings impoundment can be viewed as the following 3 stage process:

Stage 1: During this stage the wetting front advances downward through the partially saturated underlying foundation material. Above the wetting front the material may or may not be saturated.

Stage 2: When the wetting front contacts either an impermeable stratum or the phreatic surface of an aquifer, a groundwater mound will develop and rise toward the impoundment. Stage 2 represents the stage during which the mound is developing.

Stage 3: If the groundwater mound establishes contact with the impoundment, saturated seepage occurs through a mound whose height is defined by the impoundment.

Stage 1 seepage is the infiltration process discussed in Section III.6.2. Stage 2 seepage will be initiated when the wetting front reaches a phreatic surface or an impervious stratum. In many cases the cover will be more permeable than the underlying tailings and Stage 2 seepage will occur when the wetting front reaches the interface between the cover material and the tailings. Although, mounding will be initiated in the cover materials, the wetting front will continue to advance into the tailings as unsaturated flow.

If the cover material is less permeable, or has a similar permeability to the tailings, then Stage 2 seepage in the tailings will occur when the wetting front reaches the phreatic surface, or a less pervious layer such as a liner or underlying bedrock material. Stage 2 seepage is illustrated in Figure III.6-3.

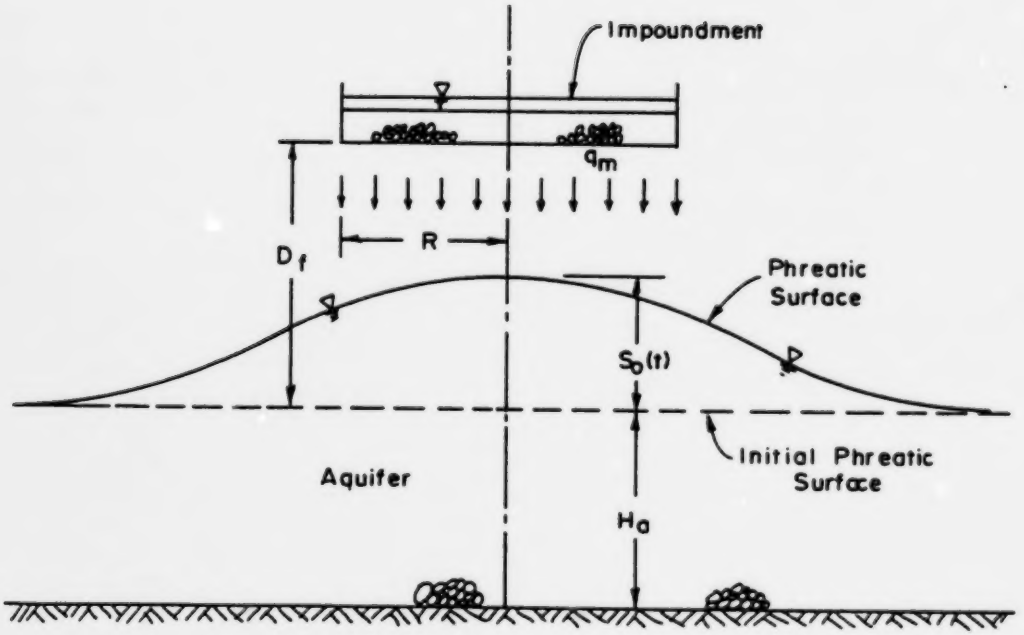
The Green-Ampt method discussed earlier in Section III.6.4, provides a simple method for determining one dimensional Stage 1 seepage through a tailings impoundment. A detailed solution of the Green-Ampt method for a two layer profile is outlined in Appendix III.6.4.

A more exact method based on a solution of the physically based Richard's Equation is presented in Appendix III.6.2. Klute and Heermann (1978) used a similar approach to model water movements in uranium mill tailings profiles. However, this type of approach is too complex for general application. The approach is data intensive and the computerized analytical solutions are susceptible to instability problems.

McWhorter and Nelson (1979) used a solution of the Green-Ampt Method to predict Stage 1 seepage out of a tailings impoundment. They also present a solution for Stage 2 mounding in the underlying foundations. Details of their solution are presented in McWhorter and Nelson (1979).

III.6.6 FRONTAL SEEPAGE FOLLOWING ABANDONMENT

In the previous sections consideration has been given to infiltration, seepage and mounding associated with recharge of water into the reclaimed tailings impoundment. Consideration will now be given to the case where there is little or no recharge and it is required to estimate the depth of movement of previously stored seepage water.



SCHEMATIC CROSS-SECTION OF GROUND WATER MOUND DURING STAGE II

FIGURE III.6-3

McWhorter (1985) notes that a distinction should be made between the "seepage front" and the "wetting front". The seepage front is the depth to which nonreactive pollutants have been carried by the seepage. McWhorter defines the "wetting front" as the coordinate of a plane located in the zone of variable content such that an exact water volume balance is obtained. For uniform antecedent water content, the depth of the wetting front is calculated by:

$$Z_{wf} = qt/(\theta_f - \theta_i) \quad (6.28)$$

where, Z_{wf} is the depth of the wetting front, q is the seepage rate per unit area, t is the duration of seepage, θ_f is the volumetric content in the wetted zone, and θ_i was the antecedent water content.

The depth of the seepage front is calculated as:

$$Z_{sf} = qt/\theta_f \quad (6.29)$$

where Z_{sf} is the depth of the seepage front and all the other terms are the same as above. If there is no recharge to the system the final depth of the seepage front following redistribution of pore pressures to hydrostatic conditions can be determined by:

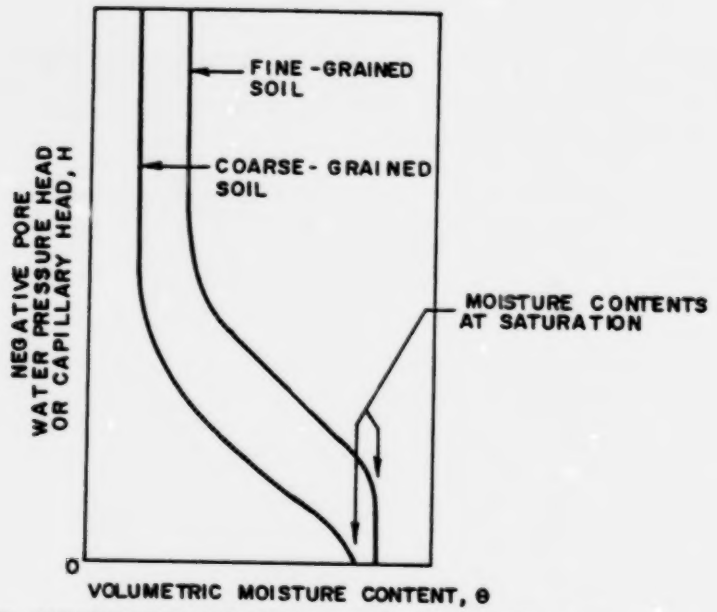
$$Z_{fnl} = V/\theta_i \quad (6.30)$$

where Z_{fnl} is the final seepage depth, V is the available depth of seepage water (volume per unit area) and the other terms are as defined previously. These relationships are illustrated in Figure III.6-4.

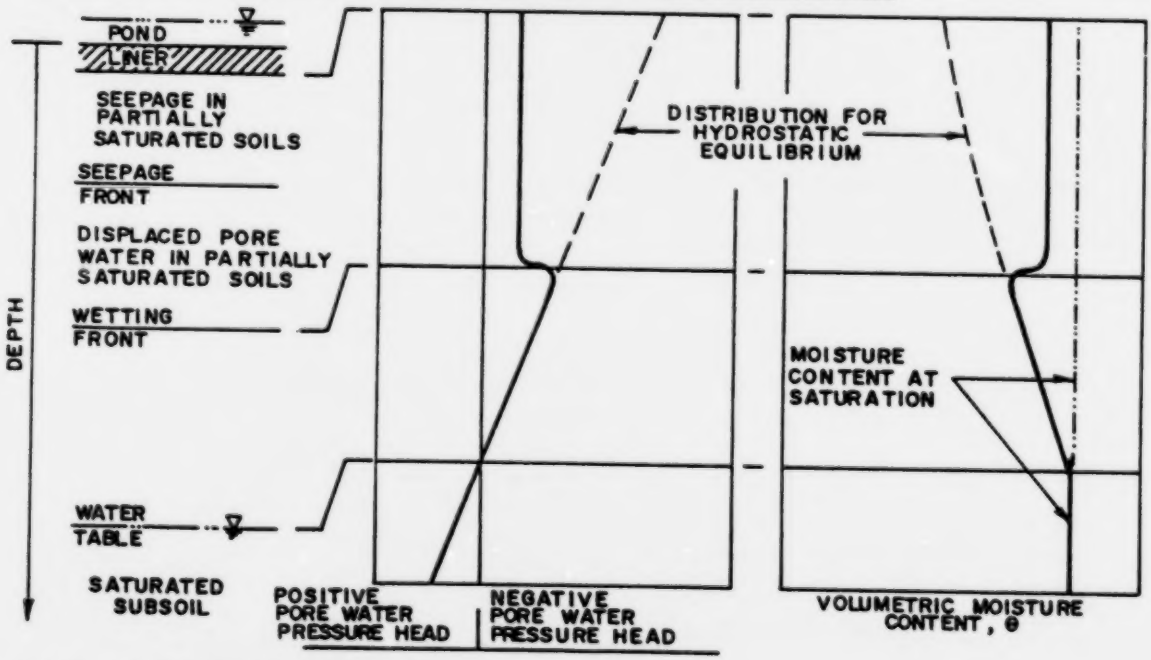
It might be expected that following reclamation, recharge of water into the tailings will be relatively small and that Equation 6.29 will provide good initial estimates of the final depth of penetration of the seepage front. The procedures are illustrated by the following simple example:

It is required to determine the final depth of the seepage front in the foundation materials under a tailings impoundment. The wetting front is currently located 5 m below the base of the impoundment. Water is also mounded in the tailings 10 m above the base. Through permeability tests and previous monitoring of water levels it has been established that the average seepage rate into, and through, the foundation is about 0.5 m per year. The drained (hydrostatic) water content in the foundations is 0.2 and the field saturated water content is 0.5. It has been estimated that the drainable water content of the saturated tailings is 0.25.

It is first necessary to determine the depth of available seepage water per unit area. In the tailings the available depth of water is 2.5 m (10 m times 0.25) and in the foundation materials the available depth is 1.5 m (5m times (0.5-0.2)). Therefore, the total depth is 4.0 m. By using Equation 6.29, the final depth of the seepage front below the tailings impoundment base is 20 m (4.0/0.2). It is also determined that it would take 8 years for the seepage front to reach this depth (4.0 m divided by the seepage rate of 0.5 m per year).



I. CAPILLARY HEAD-MOISTURE CONTENT RELATIONSHIPS



II. RELATIONSHIPS BENEATH SEEPING POND

NOTE: The above figures adapted from concepts outlined in McWhorter and Nelson (1979) and McWhorter (1985)

TYPICAL UNSATURATED HYDRAULIC PROPERTIES FOR A HOMOGENEOUS SOIL

FIGURE III.6-4

III.6.7 LEACHING AND SOLUTE TRANSFER PROCESSES

Detailed studies have been made of the hydrogeochemical changes during contaminant transport of uranium tailings seepage waters generated by leaching of tailings (for example, Morin et al (1982); Dave and Lim, (1982); Dubrovsky et al, (1985).

As a result of such studies, a degree of agreement has been found between the hydrogeochemical behavior in acid leach uranium tailings facilities in both Canada and the United States. Reviews of the overall processes involved in these hydrogeochemical systems are given in Cherry et al (1982) and Shepherd and Brown, (1982).

Some of the principal processes affecting solute transfer which have been identified in uranium tailings leachates at Elliot Lake are given in Table III.6-1, (modified from Cherry et al, (1982) to incorporate potential effects), and include:

o	dissolution reactions:	dissolution of calcite dissolution of siderite
o	replacement reactions:	siderite for calcite
o	precipitation reactions:	precipitation of gypsum precipitation of iron hydroxides precipitation of aluminum hydroxides precipitation of alumino-silicates precipitation of alunite precipitation of amorphous silica
o	ion exchange:	cation exchange removing hydrogen ions and reducing acidity and (probably) releasing sodium.

Reactions considered less important by Cherry et al (1982) include the formation of other sulphates, such as jarosite, (K, Fe, $(SO_4)_2$, $(OH)_6$), which has been found to be significant in oxidizing environments, Dubrovsky et al (1985).

Most of these reactions are invoked to explain the hydrogeochemical regime for the passage of acidic seepage generated by oxidation and leaching of the tailings through a neutralized tailings mass, a calcite-bearing soil or a carbonate-bearing (bicarbonate) groundwater. Such reactions may not be relevant to the behaviour of tailings seepage after acid generation in the system ceases. Critical to many of the reactions, however, is the position of the water table i.e. whether the reactions will occur in the aerobic, unsaturated vadoze zone or in the almost anaerobic, saturated zone in the tailings mass.

In addition, (for Elliot Lake and other pyritic tailings) once all pyrite has been oxidized, a significant proportion of the acid generation potential will be removed, although this may take many decades. Some acid generation will continue from hydrolysis reactions with aluminum-bearing

TABLE III.6-1

POTENTIAL GEOCHEMICAL PROCESSES FOR LEACHING OF URANIUM TAILINGS AND SOLUTE TRANSFER

Process	Reaction	Comment
Calcium dissolution	$CaCO_3 + H^+ \rightarrow Ca^{2+} + HCO_3^-$	If dolomite present similar reaction possible
Siderite formation	$CaCO_3 + Fe^{2+} \rightarrow FeCO_3 + Ca^{2+}$	Siderite does not occur in aerobic systems
Siderite dissolution	$FeCO_3 + H^+ \rightarrow Fe^{2+} + HCO_3^-$	Causes acid neutralization even when calcite is removed
Gypsum precipitation	$Ca^{2+} + SO_4^{2-} \rightarrow CaSO_4 \cdot 2H_2O$	Ca provided from calcite dissolution or ion exchange
Iron hydroxide precipitation	$4Fe^{2+} + O_2 + 10H_2O \rightarrow 4Fe(OH)_3 + 8H^+$	Limited by the availability of oxygen in the system
Aluminum hydroxide precipitation	$Al^{3+} + 3H_2O \rightarrow Al(OH)_3 + 3H^+$	Gibbsite or amorphous aluminum salt
Alumino-silicate precipitation	$2Al^{3+} + H_2O + 2H_4SiO_4 \rightarrow Al_2Si_2O_5(OH)_4 + 6H^+$	Formation of halloysite or similar aluminum salt
Alunite precipitation	$K^+ + 3Al^{3+} + 2SO_4^{2-} + 6H_2O \rightarrow KAl_3(SO_4)_2(OH)_6 + 6H^+$	Solubility constraint at low pH
Amorphous silica precipitation	$H_4SiO_4^0 \rightarrow SiO_2(an) + 2H_2O$	Solubility depends on major element chemistry
Cation exchange	$H^+ + M(solid) \rightarrow M^+ + H(solid)$	H^+ competes with major cations for exchange sites

minerals, discussed by Dubrovsky et al (1985). This gives the basis for aluminum precipitation-influenced seepage composition. On a longer time scale, perpetual leaching of tailings with low TDS, slightly acidic infiltration, generated by rainfall or snowmelt, will control the composition of pore water. In the latter cases, the following solution reactions could affect the behavior of the barrier materials in the longer term, due to resolution of precipitated secondary minerals:

o	resolution of gypsum	-	controlled by the solubility of calcium sulphate (SO_4 - about 1500 mg/L in pure water).
o	resolution of aluminum, hydroxides, alumino-silicates, and alunite	-	probably very slow and controlled by the low solubility of alunite in water, in the presence of sulphates, and which may be pH dependent.
o	resolution of silica	-	controlled by the equilibrium pH of the system: i.e. slow solution in acidic leaching over the long term

Based on the above description of potential hydrogeochemical reactions in uranium tailings seepage during transport, each cover or embankment material type will be examined in terms of its reactions with the generic tailings seepage compositions in TMV.

III.6.8 SALT MIGRATION

The importance of upward salt migration in tailings is very difficult to quantify from a reclamation - revegetation viewpoint in the Elliot Lake and northern Saskatchewan mining areas. Practical reclamation experience from several mining areas (Noranda, P.Q., Red Lake, Ontario, Bathurst, N.B.) strongly suggests that upward salt migration is solely a phenomenon associated with non-revegetated tailings areas. Where tailings have been direct seeded, the phenomenon has not been recorded as being problematic. Extreme variability in tailings chemical properties is, however, an established factor. It would, therefore, be improper to unequivocally dismiss upward salt migration as a potential reclamation problem in future reclamation programs whether in the Elliot Lake, Northern Saskatchewan or other regions of Canada.

Not all tailings areas can be direct seeded. Physical and chemical properties of a tailings area may prevent that type of surface treatment. Conflicting reports (most as personal communications) have been received by the authors as to the occurrences and intensity of upward salt migration into overburdens placed over tailings. Since upward salt migration will be governed by type and thickness of overburden, and by precipitation and temperature patterns in any one year, the variability in reporting is not unexpected.

Where rip-rap alone is a required method of surface protection against wind and water erosion, care must be taken that the rip-rap does not contain

soil-size material. It should be composed of clean open voids from surface to depth of placement. The presence of soil-size material could result in upward migration of salts, by capillarity, to the rip-rap surface. Therefore, rip-rap should be prepared as a construction material or, selected with care from mine waste-rock sources.

III.6.9 PRECIPITATION AND CLOGGING POTENTIAL OF UNDERDRAINAGE

Physically, the drainage/underdrainage system of a uranium tailings impoundment is a series of pore water removal conduits designed to enhance the stability of the structure. Chemically, the drainage/underdrainage system can be considered a transition zone between the pore water composition within the tailings mass and the external environment.

The transition zone becomes, therefore, a zone of reaction or potential reaction. This is because the chemical equilibria or quasi-equilibria that exist in tailings pore water are not necessarily stable in the changed environment of the drain.

The net result of changing equilibria are ex-solution and precipitation reactions. If reactions of this nature persist, there is a potential for the build-up of precipitate within the intergranular pore spaces of the drainage material or in drainage pipes. Under extreme circumstances, this can result in blockage or clogging of the drain, which consequently can have a negative effect on the stability of the tailings facility.

In general, the chemical species found in tailings pore water that have a "precipitation potential" in a drainage system include;

- o calcium salts - forming carbonate (calcite) or sulphate (gypsum)
- o iron salts - sulphates (jarosite), carbonates (siderite) or hydroxides
- o manganese salts - forming carbonates (rhodochrosite) or oxides (pyrolusite), and
- o silicon - forming silica (quartz)

Given that these four groups of compounds are varyingly soluble in tailings pore water, their actual concentration under any individual group of circumstances can be mainly considered a function of:

- o pH value - relative acidity/alkalinity
- o redox potential (eh)
- o temperature

Clearly, the ionic strength of the solution, (the potential of changing solubilities by such processes as ion pairing), will have an effect also, particularly in higher total dissolved solids (TDS) solutions.

In uranium tailings, it is anticipated that problems will occur mainly when relatively acidic, high TDS pore water or chemically reduced pore waters are discharged into the drains. With neutral, alkaline, or relatively chemically oxidized pore water, the general shift in chemical equilibrium is either small or in the direction of increasing (and not decreasing) solubility of dissolved species.

The combination of chemically acidic and/or reduced pore waters is most prevalent in the pyrite-bearing, Ontario tailings in the Elliot Lake area. The chemistry of such tailings has been discussed in previous technical memoranda (TMI-I.3.4, TMII-II.3.3). With pH as low as 2, TDS values up to 40,000 mg/L, sulphate up to 25,000 mg/L and iron levels up to 15,000 mg/L, there is significant potential for the loss of iron and sulphates from solution if the pH and/or the eh of the pore water changes on exiting to the facility drainage system.

The prediction of precipitation and clogging of drainage can be accomplished in terms of:

- identification of the potential for precipitation; and
- assessment of the mass of precipitate.

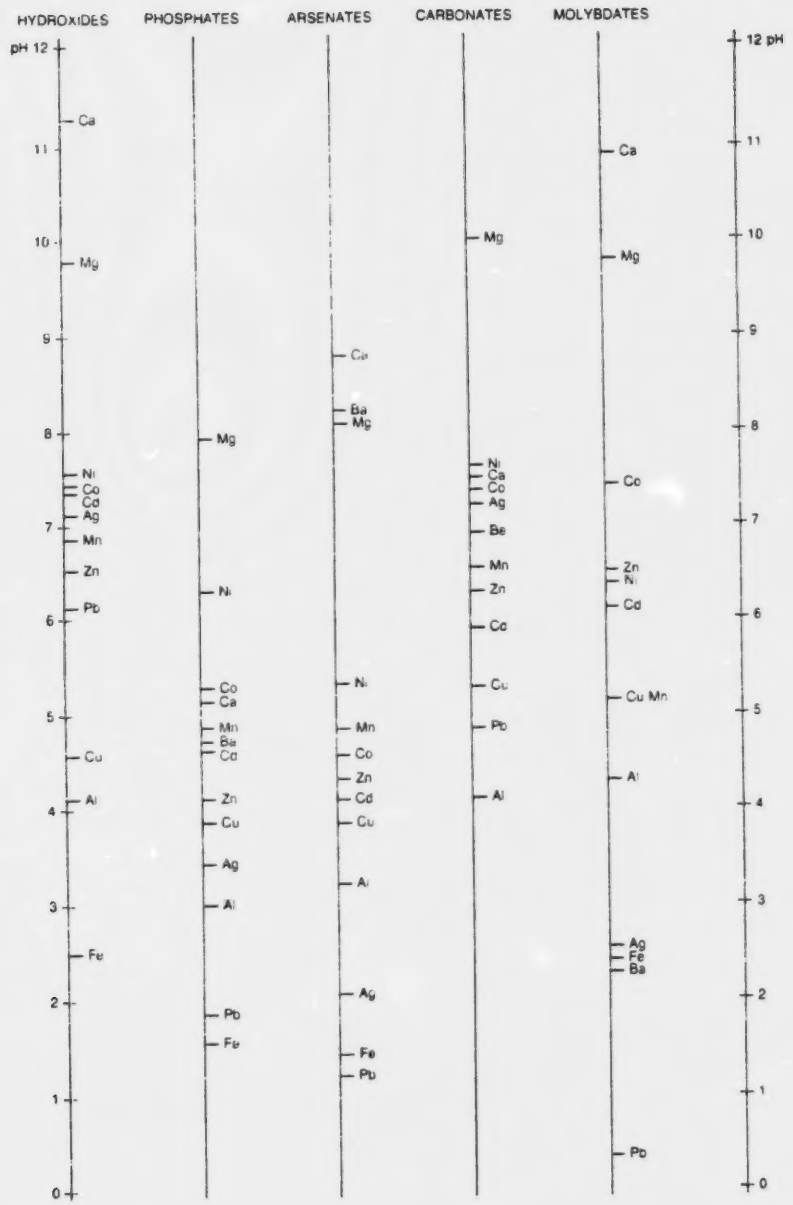
Methodology to achieve the above can be considered at a number of levels. Principal methods include:

- examination of complete chemical data by an experienced geochemist;
- tabulation/nomographs of precipitation reactions;
- equilibrium models.

The first two approaches serve only to identify the possibility of precipitation but are, at best, educated guesses as to the magnitude/extent of the problem. However, an aqueous geochemist, familiar with equilibrium will probably be able to make a basic judgement of the potential for precipitation having examined a complete chemical analysis of the pore water (including pH, eh and temperature data) and assessing the likely chemical environment in the drainage system.

Codification of such estimation and laboratory test data can give rise to tabulations/nomographs listing values of a particular variable at which precipitation would occur. An example is shown in Figure III.6-5 for pH versus anion/salt for a range of cationic species. This approach is limited to pore solutions of that salt (e.g. hydroxide) or solutions where a single anionic species is completely dominant.

A superior approach may be the use of equilibrium models. The use of such models is advocated by Johnson, (1985). In essence, such models have been used to look at chemical equilibrium in waters, WATEQ, versions 1, 2 and 3; Truesdell et al (1974), Ball et al (1979), Ball et al (1981), and the interrelationship between mineral phases, precipitates and waters, (PHREEQE; Parkhurst (1980), SUFEQL; Morel and Morgan (1972)). These models require detailed and precise chemical analysis of the majority of



APPROXIMATE pH VALUES (AT 25°C) AT WHICH PRECIPITATION OF VARIOUS METAL SALTS COMMENCES
(SOURCE: CHIEF CHEMIST, MODDERFONTEIN FACTORY, AECI LTD. PERSONNEL COMMUNICATION (1982))

FIGURE III.6-5

components in the system to be truly effective and to give reliable predictions.

However, models have been used successfully by workers on the UMTRAP program in the United States for prediction of seepage input of groundwater system and hence their applications to uranium tailings impoundments, in general, has some validity. There is no apparent record of their use for predictions of precipitation in drainage systems, although the application there is obvious.

III.7.0 SLOPE STABILITY

III.7.1 TYPES OF EMBANKMENTS

Embankments used to retain uranium tailings can be divided into three broad categories.

(i) Natural Embankments or Retainment Features

Almost all tailings impoundment sites in Canada are located in natural topographical depressions. The majority of the rims to such natural basins is provided by topographic features or natural embankments. Where tailings are deposited into lakes, then the entire rim is formed by such natural embankments. These natural embankments are usually of very low slope angle and, having existed in their present form for a very long time, are often demonstrably stable for a very long time into the future.

There may be circumstances where such natural embankments do not have large reserves of strength and long term instability could occur. Examples are:

- where the natural embankment has a limited width, such as a narrow ridge line formed by rock, glacial side moraine, ancient river terrace gravel, etc.

- where the natural embankment is being actively steepened by river erosion or other geomorphological processes

- where the embankment has been created or steepened by man made excavations such as when an open pit is used to retain tailings.

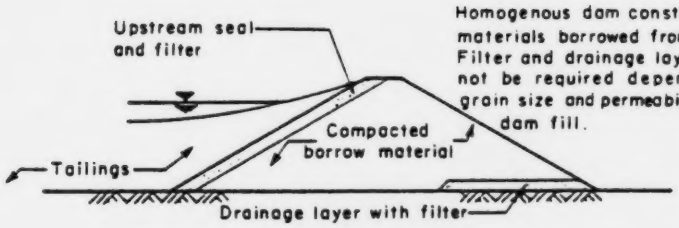
(ii) Rolled Earth Embankment

These are commonly used to fill in any low spots in the natural topographic basins used for uranium tailings impoundment development. Common types are (Fredlund, 1985):

- Homogeneous Cross-sections
- Thin Core Cross-sections
- Zoned Cross-sections
- Rock-Fill Cross-sections

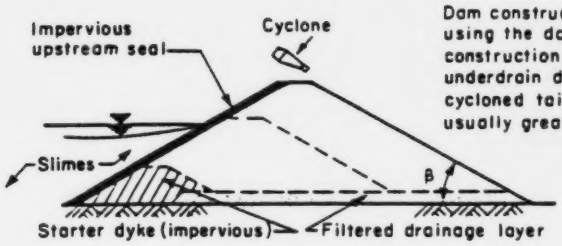
Figure III.7-1 and -2 show examples of several of the above types of cross-sections in their application to waste management. Figures III.7-1a and -1b show homogeneous type cross-sections. These sections often make nominal use of filters and seals. Figure III.7-1c shows a Rock-Fill type cross-section with a thin core and gradational filters. Figures III.7-2a and -2b show thin core type cross-sections and emphasize design procedures to control seepage losses. Figures III.7-2c and -2d illustrate the use of pumping and injection systems to control seepage from the pond.

Homogeneous type cross-sections have the entire cross-section predominantly of one soil type. This is generally the best type of cross-section when the height of the dam does not need to exceed 50 feet (17 metres). This structure is least costly and is easy to construct.



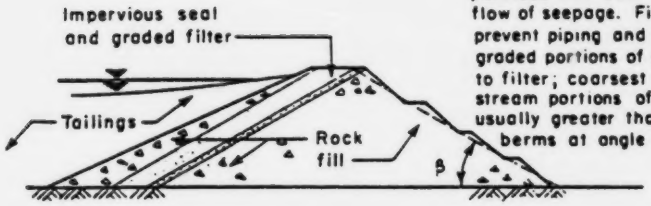
Homogenous dam constructed of compacted materials borrowed from nearby source Filter and drainage layers may or may not be required depending on relative grain size and permeability of tailings and dam fill.

(a) Homogenous dam



Dam constructed of cycloned tailings using the downstream method of construction. Requirements for filtered underdrain depend on permeability of cycloned tailings. Cotangent β usually greater than 2.0

(b) Cycloned tailings dam

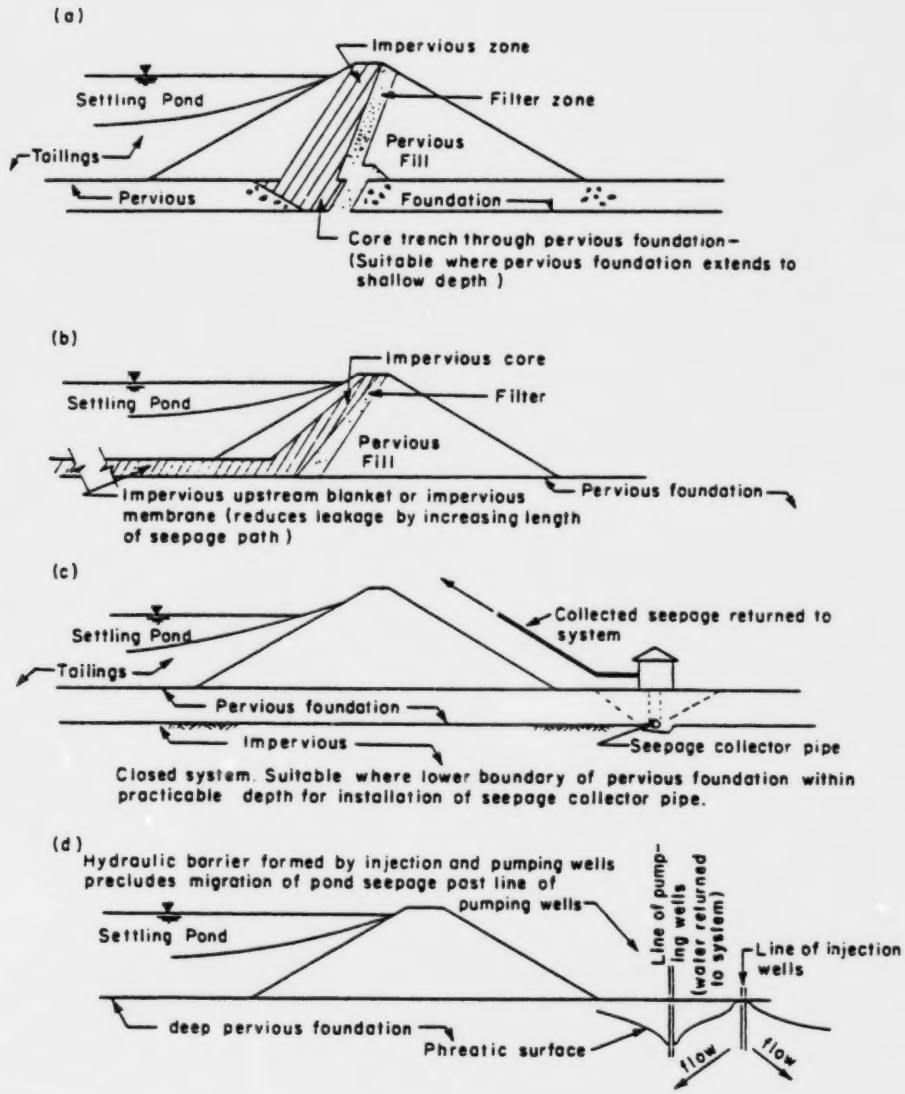


Dam constructed of waste rock or other pervious fill. Seal is required to restrict flow of seepage. Filter layer is required to prevent piping and internal erosion. Finely graded portions of rock fill placed adjacent to filter; coarsest rock placed within downstream portions of section. Cotangent β usually greater than 1.5. Slopes between berms at angle of repose.

(c) Waste rock dam

EXAMPLES OF TAILINGS DAMS ON IMPERVIOUS FOUNDATIONS
(FROM PIT SLOPE MANUAL, CHAPTER 9, 1977)

FIGURE III.7-1



EXAMPLES OF TAILINGS DAMS ON PERVIOUS FOUNDATIONS
ILLUSTRATING SEEPAGE CONTROL
(FROM PIT SLOPE MANUAL, CHAPTER 9, 1977)

FIGURE III.7-2

Any dam over approximately 25 feet (8 metres) in height should have some form of drain incorporated into the Homogeneous cross-section. Figure III.7-3 shows examples of toe and horizontal drains. The toe drain draws down the line of seepage and allows the exit of foundation seepage water. The purpose of the drains is to reduce pore pressures at the toe of the embankment and the potential for a piping type of failure.

Thin core type cross-sections can be divided into two types; namely, vertical thin cores and sloping thin cores. Examples are shown in Figure III.7-4. Thin cores are often used when there is a scarcity of impervious material. Also, a wet climate may encourage the use of a thin core cross-section since it reduces the amount of clay that must be handled. Theoretically, the core could be only a few feet in thickness to prevent excessive leakage. However, other factors must also be taken into account. Thicker cores are less susceptible to cracking and subsequent piping. It is difficult to use construction equipment to place a zone less than 8 feet in thickness. 'Past Precedent' in a particular geographic region should also play an important role in the design thickness of the core. In general, a core thickness equal to 30% of the head at any point is considered acceptable. Core thicknesses of 15 to 25% of the head are considered to be very thin cores. These must be adequately protected by using filters.

A zoned type cross-section is essentially a thick core dam. An abundance of relatively impervious core material usually dictates this type of a cross-section. In western Canada, there is an abundance of glacial till and this material is suitable for the thick central core.

Rock-fill type cross-sections have a relatively thin core and substantial rock-fill shells. The interfaces between the rock-fill and the thin core must be protected with numerous filters. The filters are the key to the successful performance of this type of dam.

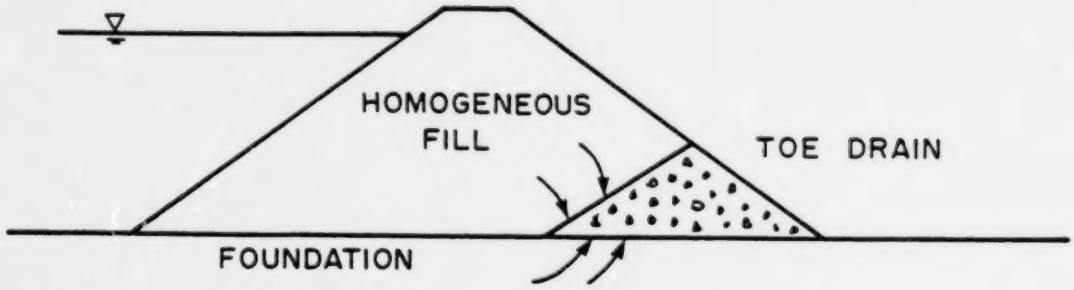
(iii)Hydraulic Fills and Dams

The tailings solids can often be used to construct the dam retention structure. This greatly reduces costs, allowing the dam to be built as the mining operation proceeds. Where the tailings consist mainly of sand to silt-size material they can readily be placed as a hydraulic fill.

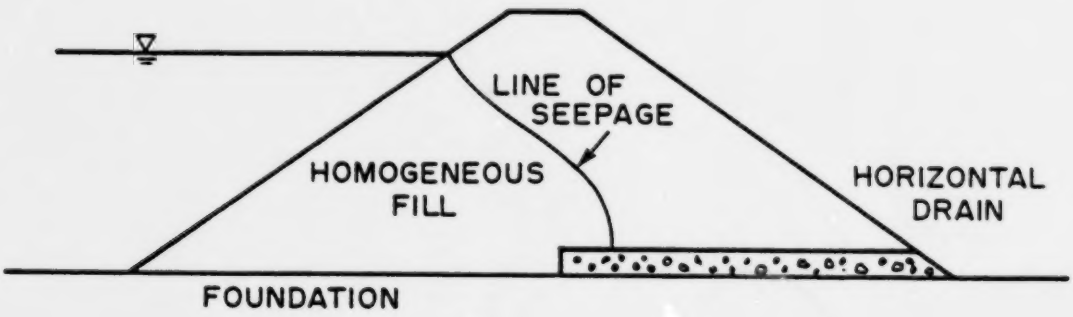
The types of placement techniques can be categorized as follows:

- upstream method
- downstream method
- fixed centreline method

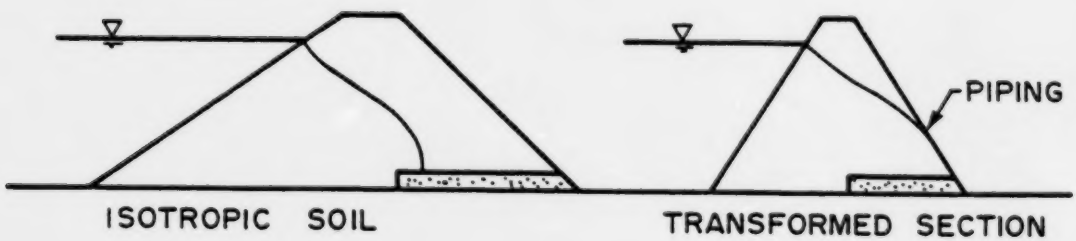
These three methods are shown in Figure III.7-5. Each of the methods may require some initial preparation of the foundation and the placement of a drainage system. A starter dam may or may not be built. The tailings are sometimes cycloned to obtain the coarser fraction for the construction of the dam. Compaction may or may not be used on the dam.



3a) Toe Drain in a Homogeneous Cross-Section



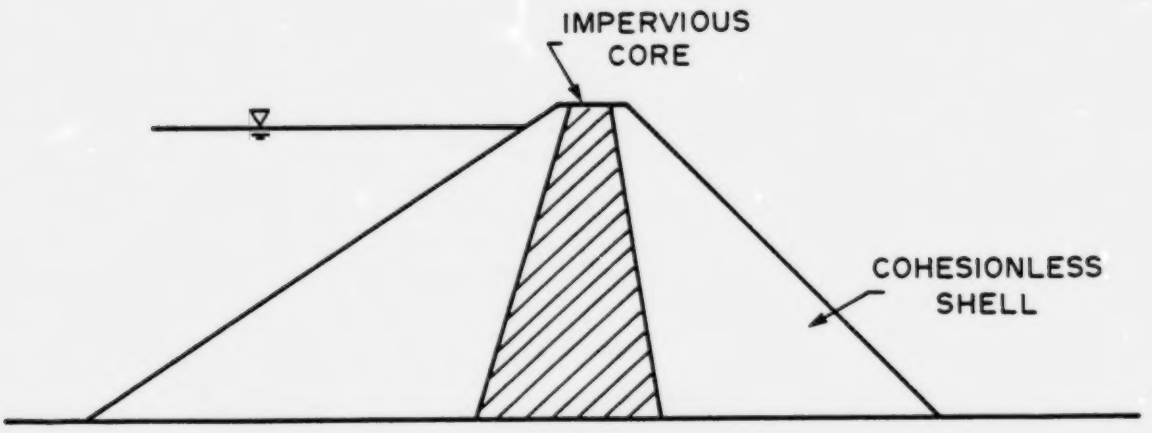
3b) Horizontal Drain in a Homogeneous Cross-Section



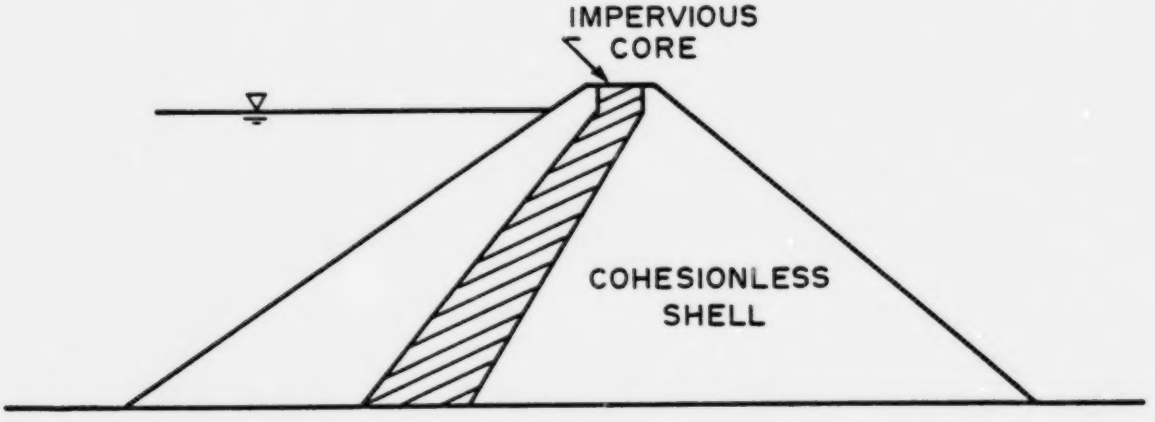
3c) Transforming Cross-Section to Check for Line of Seepage Exit

USE OF DRAINS ON HOMOGENEOUS CROSS-SECTIONS
(FROM FREDLUND, 1985)

FIGURE III.7-3



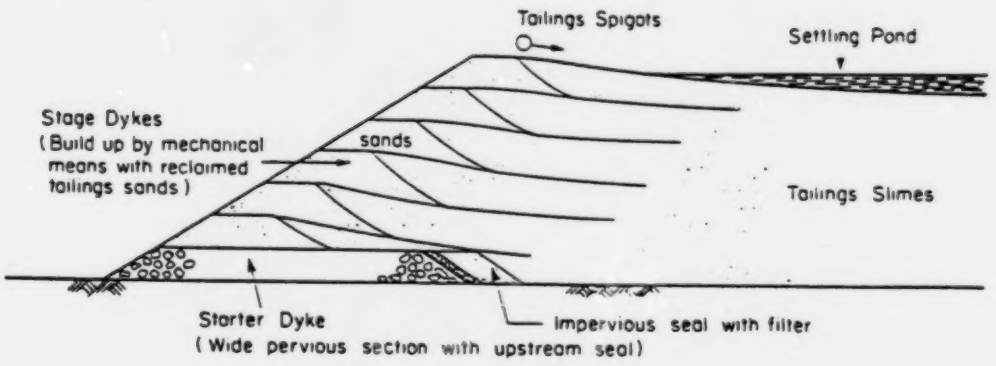
4a) Vertical Thin Core Dam Cross-Section



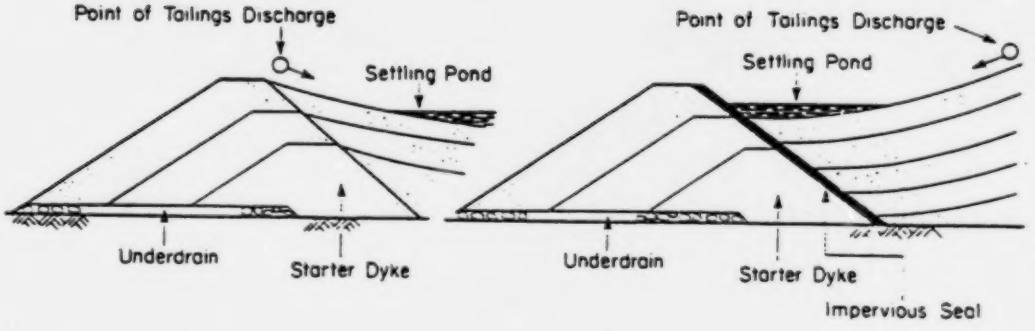
4b) Sloping Thin Core Dam Cross-Section

EXAMPLES OF THIN CORE CROSS-SECTIONS

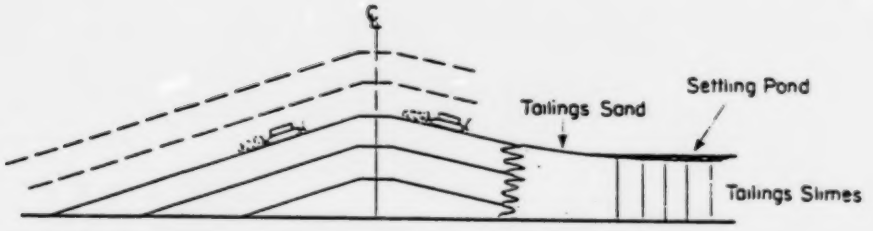
FIGURE III.7-4



UPSTREAM METHOD
(of stage construction)



DOWNSTREAM METHOD
(of stage construction)



FIXED CENTRELINE METHOD
(of stage construction)

TAILINGS EMBANKMENT CONSTRUCTION METHODS
(FROM PIT SLOPE MANUAL, CHAPTER 9, 1977)

FIGURE III.7-5

The upstream method is the least costly technique. Therefore it is usually the first method considered. However, it is most susceptible to instability problems. It may be ruled against as a possible design due to: i) the seismicity of the region, ii) the toxicity of the waste material, iii) the high risk associated with the population living below the dam or, iv) other reasons related to stability and seepage. It is often difficult to keep the line of seepage from the settling pond from exiting on the downstream face of the dam. Such an exit condition could lead to a piping type failure.

The downstream method allows the design and construction of a more stable dam, thereby reducing 'risk'. Borrow or dry-waste material can be placed, making many alternative types of embankments possible.

The centreline method is essentially a variation of the downstream method where the rate of rise of the embankment is carefully matched with the rate at which material is placed in the impoundment in order to eliminate the risk of failure towards the interior of the impoundment. This allows a considerable saving in the volume of embankment material.

Both the centreline and downstream construction methods permit the inclusion of engineering features such as toe drains and compaction of the layers of tailings.

Some of the embankments in the Elliot Lake uranium producing area were constructed of tailings using the centreline or downstream methods.

III.7.2 COMMON TYPES OF FAILURES

Four common types of embankment mass movement failures are illustrated in Figures III.7-6 to -9.

The planar shear mode is observed both during and after construction. A common cause of such failures is induced pore water pressures at a shallow depth below the embankment surface. These pressures may develop as a result of:

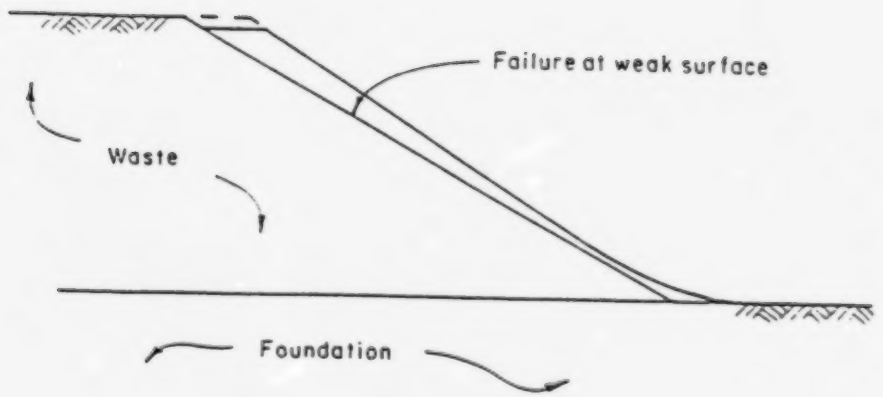
(i) ice accumulation and thawing in frost susceptible soils

(ii) rapid infiltration in wet periods in the more permeable (loosened) surficial embankment soil layers. Such sloughing of the embankment slopes is observed to occur after many years of general embankment stability.

Rotational sliding is most commonly experienced during construction. It can be induced after construction is completed if conditions along the shear surface change due to one of the following:

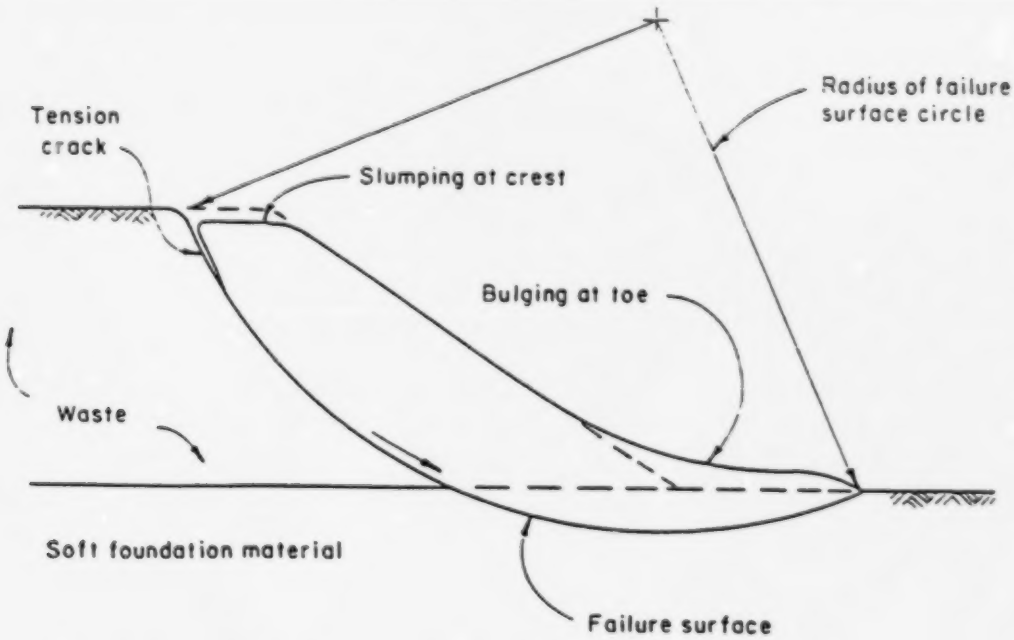
(i) a reduction of shear strength (due to leaching, weathering and decomposition) of the material along the failure surface

(ii) changed geometry of the embankment due to erosion/by stream action or piping of the toe of the slope



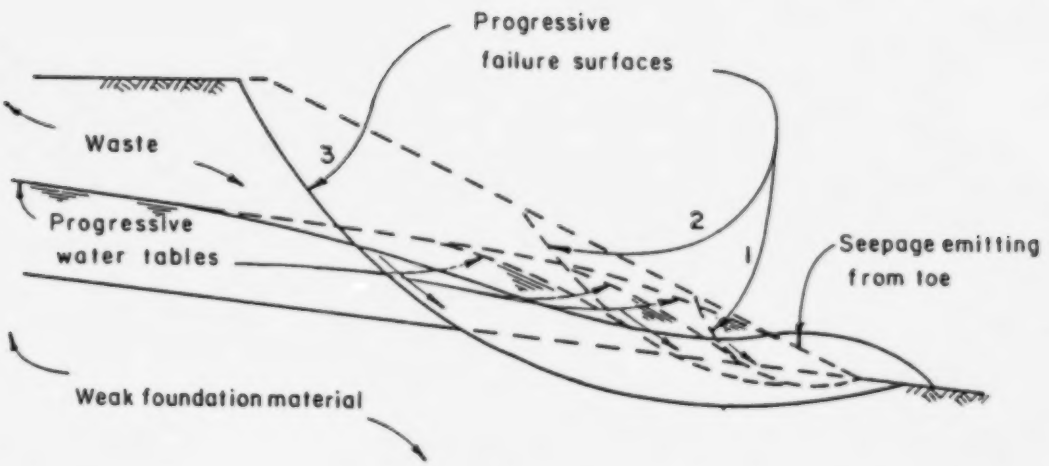
PLANAR SHEAR MODE

FIGURE III.7-6



ROTATIONAL SHEAR MODE

FIGURE III.7-7(a)



PROGRESSIVE FAILURES

CIRCULAR SLIP SURFACES FORMING A RETROGRESSIVE SERIES OF MASS MOVEMENTS
(FROM PIT SLOPE MANUAL, CHAPTER 9, 1977)

FIGURE III.7-7(b)

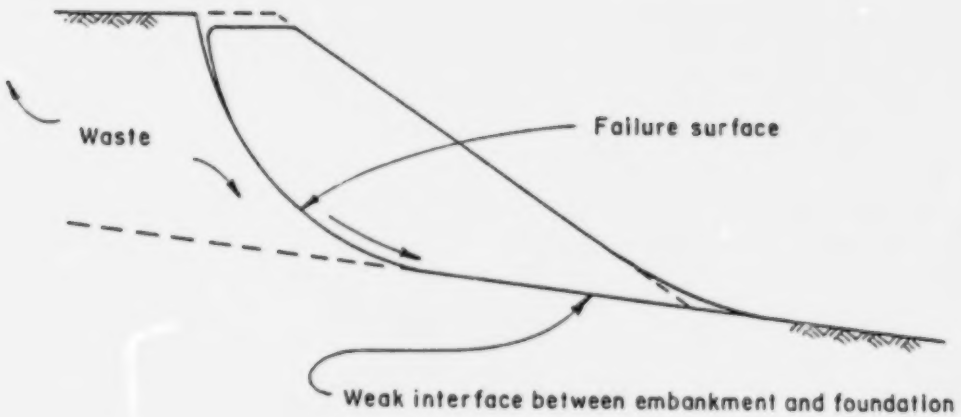


FIGURE III.7-8 EXAMPLE OF A NON-CIRCULAR OR COMPOSITE TYPE SLIP SURFACE

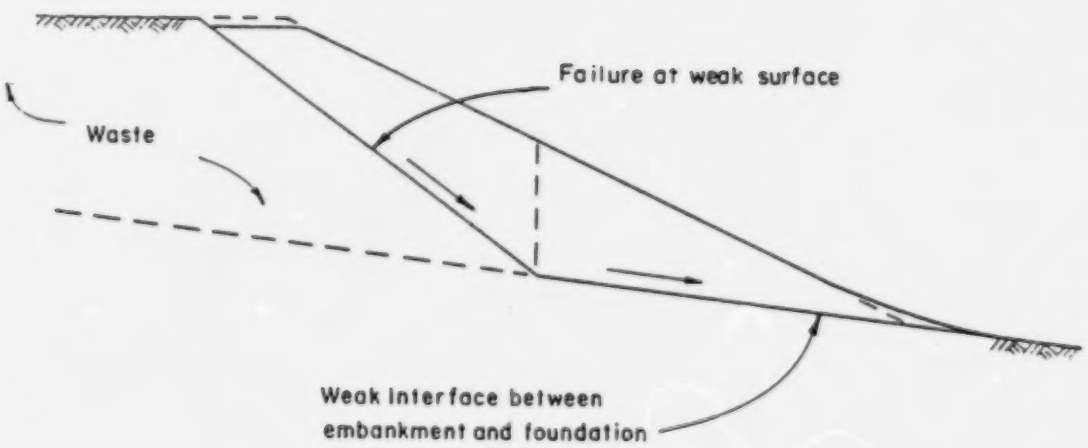


FIGURE III.7-9 EXAMPLE OF A WEDGE-TYPE OF SLIDING MASS (FROM PIT SLOPE MANUAL, CHAPTER 9, 1977)

(iii) changed pore pressure conditions along the failure surface due to changes in the phreatic surface

(iv) seismic events occur which will both induce temporary stress changes on the failure surface and may result in elevated pore pressures due to dynamic liquefaction of some of the embankment materials

Initiation of failure at the embankment toe due to one of these causes, pore pressure changes for example, can result in progressive failure as illustrated in Figure III.7-7b).

Non circular composite failure surfaces (Figure III.7-8) and wedge type failures (Figure III.7-9) result where the embankment and foundation materials have substantial strength differences resulting in more generalized failure surfaces. Failures can develop after construction is complete for the same reasons as for circular slip surfaces.

III.7.3 STABILITY ANALYSES

The limit equilibrium method of stability analysis using the method of slices (Fredlund 1985) is suitable for the evaluation of the long term stability of a uranium tailings embankment.

In this method the various potential slip surfaces are postulated. Each slip surface is divided into a series of slices as illustrated in Figure III.7-10. The shear strength along the base of each slice is determined using the equation:

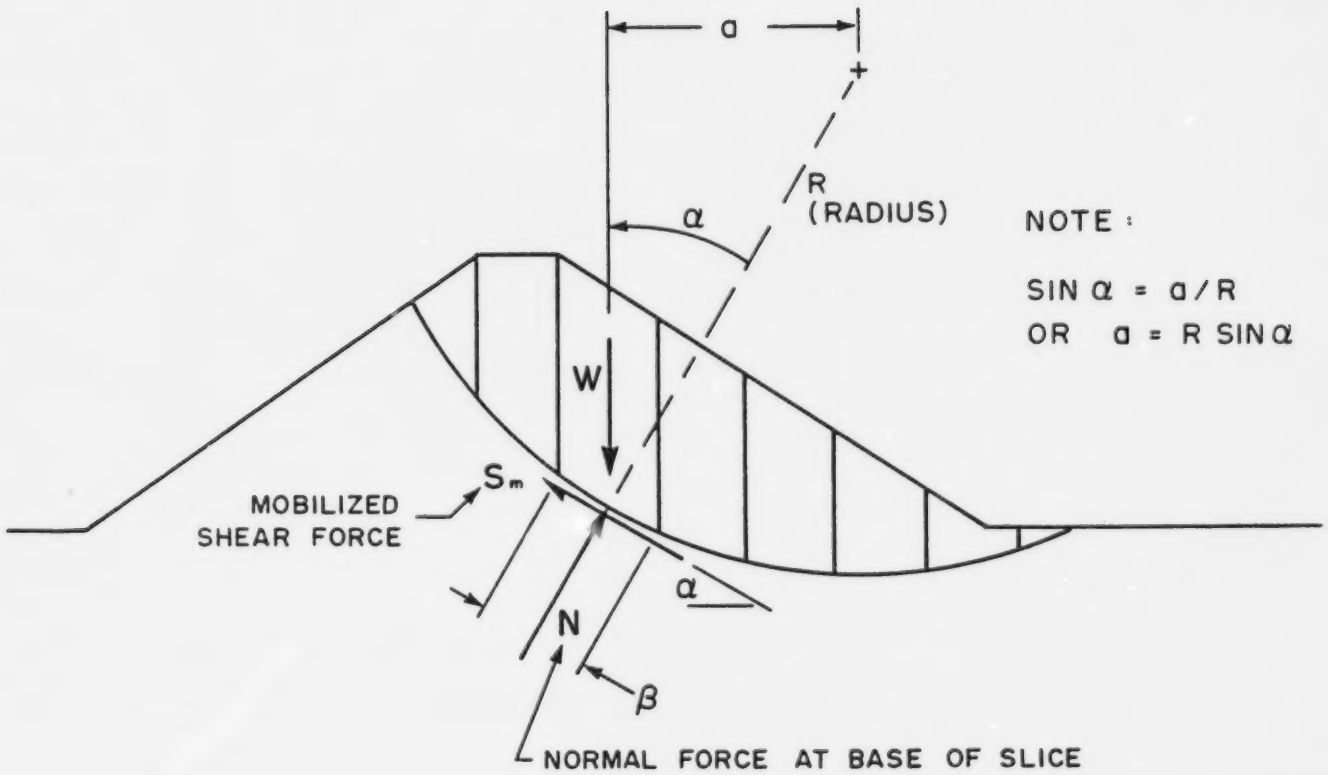
$$s = c' + (\sigma - u)\tan \phi' \qquad (7.1)$$

where
- s = shear strength on the base of that slice
- c' = coefficient of cohesion of the material at the base of the slice
- σ = normal stress on the base of the slice
- u = pore water pressure on the base of the slice
- ϕ' = effective angle of friction of the material at the base of the slice

A number of different methods, based on different assumptions, have been developed to determine the interslice forces, and therefore the stress conditions along the failure surface. The end product of each of these analysis methods is a comparison of the sum of the resisting forces (ΣRF) to the sum of the disturbing forces (ΣDF). A factor of safety against failure is determined by one of two common methods.

(i) The factor of safety (F) is taken as the ratio of the sum of the resisting forces to the sum of the disturbing forces, i.e.,

$$F = \frac{\Sigma RF}{\Sigma DF} \qquad (7.2)$$



CROSS-SECTION SHOWING THE INSCRIBED CIRCULAR SLIP SURFACE DIVIDED INTO SLICES

FIGURE III.7-10

(ii) A reduction factor (Fs) is applied to material strength parameters until the sum of the resisting forces is equal to the sum of the disturbing forces

$$1.0 = \frac{\Sigma \frac{RF}{Fs}}{\Sigma DF} \qquad (7.3)$$

A number of the most popular analytical methods are described by Fredlund, (1985). These techniques have been programmed into a number of computer codes permitting numerical analyses. The Slope-II and PC-Slope programs (Fredlund 1985) and a large number of other commercially available programs are suitable for the stability analyses of uranium tailings retention embankments.

III.7.4 EVALUATION OF INSTABILITY

III.7.4.1 Likelihood of Instability and Release Rates

Tailings retention embankments are designed and constructed to be stable at the time of abandonment of the impoundment. Instability develops as a result of long term changes which may occur. The determination of the likelihood of instability is therefore dependent on the extent of the changes that can occur. The time rate of tailings release which may occur is dependent on how quickly these changes could come about and the size of the release following failure.

The determination of the likelihood of instability requires an estimate of the likelihood of changed conditions which are sufficient to bring about failure. Possible changes are:

(i) material property changes resulting in loss of shear strength and decrease or increase in permeability,

(ii) geometric changes resulting from erosion,

(iii) phreatic surface level changes,

(iv) seismic effects

III.7.4.2 Material Property Changes

Embankment construction materials may be divided into the following broad classifications:

(i) freshly quarried rock fill

(ii) tailings

(iii) soils from local borrow sources

(iv) imported soil materials

(v) synthetic construction materials such as geofabrics

Freshly Quarried Rock

Freshly quarried rock fill will have a relatively high shear strength and permeability shortly after placement. The rate at which this rock fill will weather and break up is dependent on the nature of the rock material. The long term survivability of some rock types has been evaluated by Lindsey et al (1982). The highly silicious quartzites of the Elliot Lake region will weather slowly and may change little in a 100 to 200 year time period. The sedimentary rock waste from some of the Saskatchewan mines will weather rapidly and may decompose to soil consistency within a few years. The shear strength of argillaceous rock types may be expected to ϕ' reduce considerably during such weathering. Value changes could range from greater than 37° when freshly quarried to ϕ' less than 25° when entirely decomposed. Changes in the permeability could be even more dramatic ranging from greater than 10 cm/sec for fresh rock to 10^{-6} cm/sec for decomposed argillaceous rock materials. If the weathered products are soluble then there may be a leaching out of the rock mass volume.

Methods for the evaluation of rock durability have been proposed as a measure of its suitability as a long term rip-rap (Foley et al 1985). Test methods include:

- Bulk specific gravity
- Absorption
- Freeze-Thaw
- Sodium Sulphate Soundness
- Los Angeles Abrasion
- Schmidt Hammer
- Sonic Velocity
- Ultrasonic Cavitation
- Slake Durability

Where seepage through the rock materials carries chemical constituents which may accelerate decomposition (such as acid tailings water) the above tests may be of little relevance. For the more rapidly decomposing rock types, the slake durability test in a solution of the seepage water may be of value. Inspection of local outcrops of the weathered rock may provide a crude indication of the rate of decomposition.

If complete decomposition is anticipated in the 100 to 200 year time frame of interest then the permeability and shear strength of equivalent products can be determined by conventional laboratory shear testing methods. Stability calculations can be made using the reduced shear strengths.

Because of the relatively low slope angles normally associated with embankments on abandonment it is considered unlikely that strength reductions due to weathering alone would result in slope instability.

Tailings

Tailings, like fresh quarried rock, may weather relatively rapidly. The coarse fraction of tailings from the more argillaceous ore types (Saskatchewan) are expected to break down substantially in a 100 year

period. Silica grains from the Elliot Lake pebble conglomerate ores may show little change. The same comments apply to strength and permeability of tailings used in embankment construction as does to fresh quarried rock. Where the tailings contain leachable products (such as gypsum at Key Lake), long term leaching will remove a percentage of the tailings solids effecting shear strength, permeability and surface stability.

Soils from Local Borrow Sources

Surficial soils have been subjected for long periods to many of the weathering processes that apply to the embankment materials. They are therefore likely to be more durable and weathering resistant. Care should be exercised in evaluating whether there are products in the soils which are soluble in the tailings or leaching solutions. Leaching of carbonate rich soils by acidic seepage can increase seepage and reduce strength. This increases the potential for piping failures.

Glacial outwash sands on the Canadian shield are durable and relatively inert. Similarly, little potential exists for degradation of the mixed till materials. Organic soils (peat or muskeg) may degrade and lose mass fairly rapidly under adverse conditions.

Imported Soil Materials

Imported soils and natural admixture materials such as bentonite, must also gain equilibrium under the local conditions. These may be expected to be more susceptible to degradation than locally borrowed materials.

The durability of both locally available and imported earth liner materials has been considered by Clifton and Associates (1986).

Synthetic Construction Materials

Geomembranes (PVC, HDPE, Hypalon, etc.) are used to control seepage, mainly for ponds or reservoirs that are required to function only during the operating life of the milling facility. Synthetic fabrics are often used in embankment construction for items such as filter layers between zones of different permeability. The durability of such materials has not been established for periods approaching 100 or more years. The long term loss of such layers must be anticipated. If failure of such layers results in the piping of fines into course drainage layers then consequences of the loss of such drainage facilities must be considered.

III.7.4.3 Geometric Changes Resulting from Erosion

Because of the costs of construction, embankments are usually constructed at slope angles which are as steep as possible but which achieve overall stability against rotational slope failure. There are three potential and common causes of slope modification that occurs after abandonment.

(i) Gulley or rill erosion down the comparatively steep embankment slopes

(ii) Erosion of the toe of the slope by streams or diversions which flow along the toe of the slope

(iii) Piping at the toe of the embankment, eroding the toe

The methodology for erosion evaluation and piping are reviewed in sections 2 and 8 of this document.

III.7.4.4 Phreatic Surface Level Changes

An increase in the phreatic surface in the toe area of an embankment can make a dramatic change in the stability of that embankment. Shown in Figures III.7-11 and -12 are stability charts taken from Hoek and Bray, (1977), which may be used to determine the difference in factor of safety for two different phreatic surface locations. Earth embankments which are perfectly stable with the phreatic surface as defined in chart 1 may fail when subject to the conditions on chart 3.

Increases in the phreatic surface can be caused by:

(i) increased infiltration into the tailings behind the embankment

(ii) decreased permeability of the embankment material due to weathering or the precipitation of chemical products.

(iii) decreased permeability of the drain due to:

- weathering of drainage material
- washing in of fines into the drainage material
- chemical precipitates in the drains

(iv) freezing of the drain, drain exits and the embankment face

The potential for each of these changes are evaluated elsewhere.

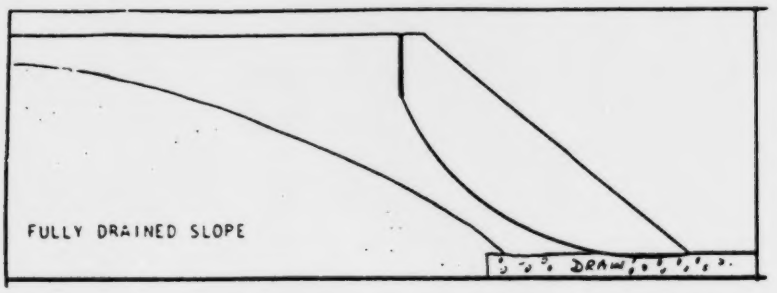
III.7.4.5 Seismic Instability

The effect of earthquake or seismic loadings is taken into account by applying a horizontal force through the centre of gravity of each slice (Fredlund 1985) in the stability analysis. The magnitude of this force is expressed as a fraction of gravity.

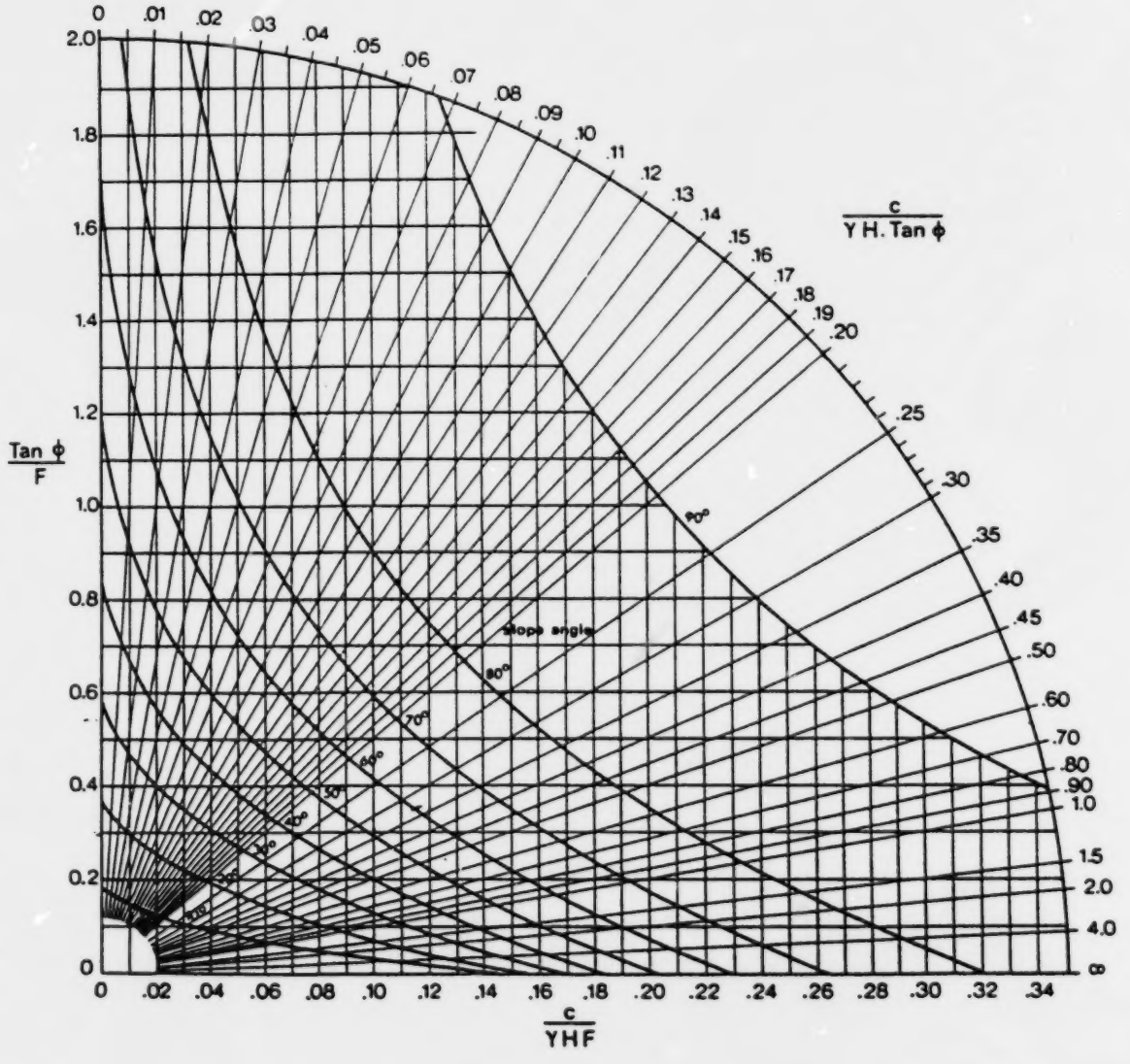
$$\text{Earthquake Force} = kW \tag{7.4}$$

where W = weight of each slice
k = seismic coefficient equal to (a/g)
a = horizontal acceleration due to the earthquake
g = acceleration due to gravity

The procedure is a simplification of the real loadings during an earthquake. However, this is the procedure most widely used in conjunction with limit equilibrium methods. The general design range for the seismic coefficient is from 0.05 to 0.25. The seismic coefficient is applied to the soil portion of any slice while the water portion, during partial submergence, is ignored.



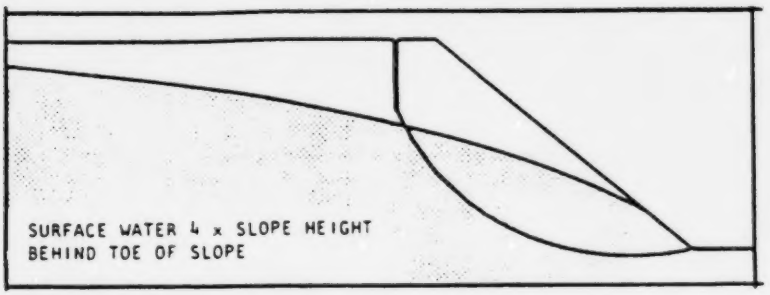
CIRCULAR FAILURE CHART NUMBER 1



SLOPE STABILITY CHART FOR FULLY DRAINED CONDITIONS

(TAKEN FROM HOEK AND BRAY, 1977)

FIGURE III.7-11



CIRCULAR FAILURE CHART NUMBER 3

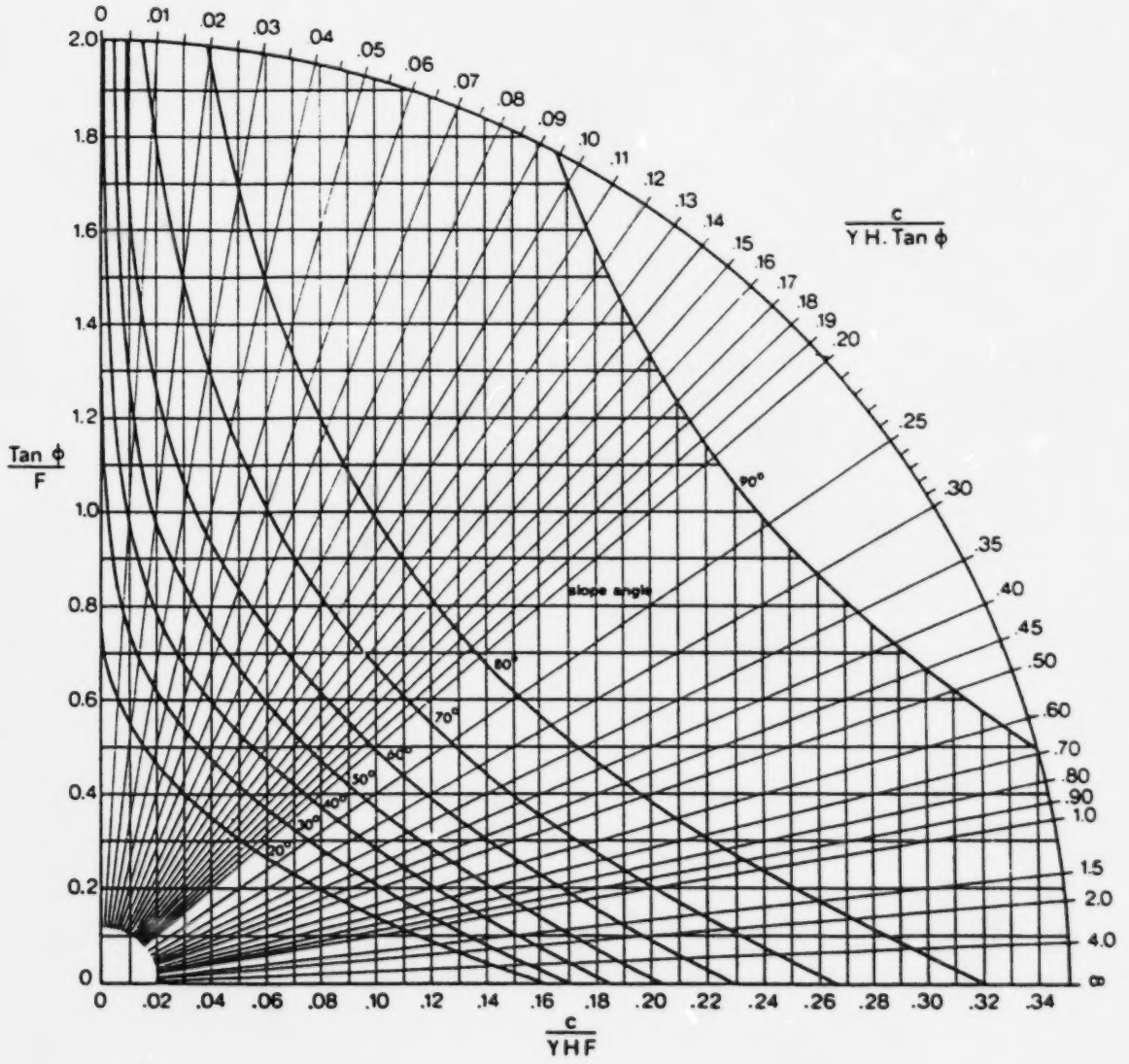
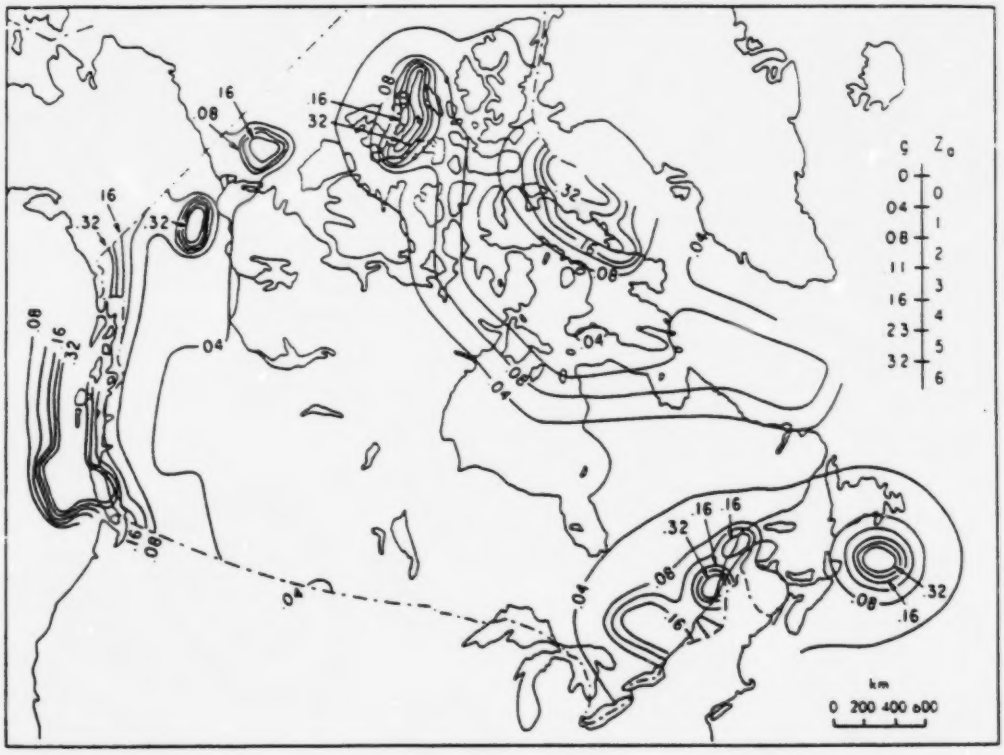


FIGURE III.7-12 SLOPE STABILITY CHART FOR RELATIVELY HIGH WATER TABLE (TAKEN FROM HOEK AND BRAY, 1977)

Note that an earthquake of a specific magnitude does not mean that an equal gravity force should be applied to the embankment. The damping effect of the soil greatly reduces the required design seismic coefficient. For example, an earthquake of 0.7g may correspond to a design seismic coefficient between 0.2 and 0.3. The above approach is referred to as a 'Pseudo-static' type analysis. Although it does not accurately model behaviour of an embankment during an earthquake, it is a necessary analysis to perform in an earthquake-prone region.

Figure III.7-13 shows the seismic values for Canada as a percentage of gravity acceleration value having a 10% probability of exceedence in 50 years. The figure shows that there is essentially no significant risk of seismic activity in the uranium producing areas of Canada.

Where loose saturated foundation soils, embankment materials, or tailings are subjected to cyclic seismic stresses, this may result in liquefaction and flow of the materials. Embankments are usually designed and constructed to prevent failure from this cause by compacting the embankment materials. In view of the low seismic activity in the uranium producing areas of Canada, this is not considered a long term stability concern. Methods for evaluation are summarized in Vick (1983).



CONTOURS OF PEAK HORIZONTAL GROUND ACCELERATION IN UNITS OF 'g' HAVING A PROBABILITY OF EXCEEDANCE OF 10% IN 50 YEARS; Z_a = SEISMIC ZONE BASED ON ACCELERATION

FIGURE III.7-13

III.8.0 SEEPAGE EROSION

III.8.1 INTRODUCTION

The failure of geotechnical structures due to the flow of water under high seepage gradients is usually described as "piping", "heave", or "internal erosion" failures. The term "seepage erosion" is much more general in describing the adverse effects of seepage and includes the failure mechanisms above. Most of the time, especially with respect to natural soil deposits, seepage erosion is a far better descriptor for this kind of failure because of the uncertainties involved in the exact failure mechanism.

This section describes seepage erosion mechanisms as they apply to uranium tailings impoundments and the approaches suggested for their evaluation.

III.8.2 POTENTIAL SEEPAGE EROSION OF URANIUM TAILINGS STRUCTURES

Seepage erosion of uranium tailings structures can occur as a result of upward flow downstream from an embankment or seepage exiting a slope under unconfined conditions. Figure III.8-1 presents a schematic of the two general modes of failure.

Seepage erosion failures due to upward flow can be classified into three modes:

(i) heave,

(ii) piping, and

(iii) internal erosion.

Each of these modes is controlled by a different set of failure mechanisms, and each is analyzed in a different fashion. Table III.8-1 presents a summary of the mechanisms and techniques of analysis for each failure mode as well as some examples for each as they apply to uranium tailings impoundments.

Presently available technology allows a more rational approach to the evaluation of seepage erosion due to upward flow than to unconfined flow.

III.8.3 SEEPAGE EROSION MECHANISMS AND ANALYSIS-UPWARD FLOW

Development of design methods to investigate the safety of geotechnical structures against seepage erosion have followed two main directions, namely: the global gradient approach and the exit gradient approach. Calculated gradients are compared to an allowable global gradient (or critical exit gradient) to establish the relative stability of structures against seepage erosion. Considerable attention has also been given to the analyses of heaving failures. Both of these can be applied to seepage erosion due to upward flow.

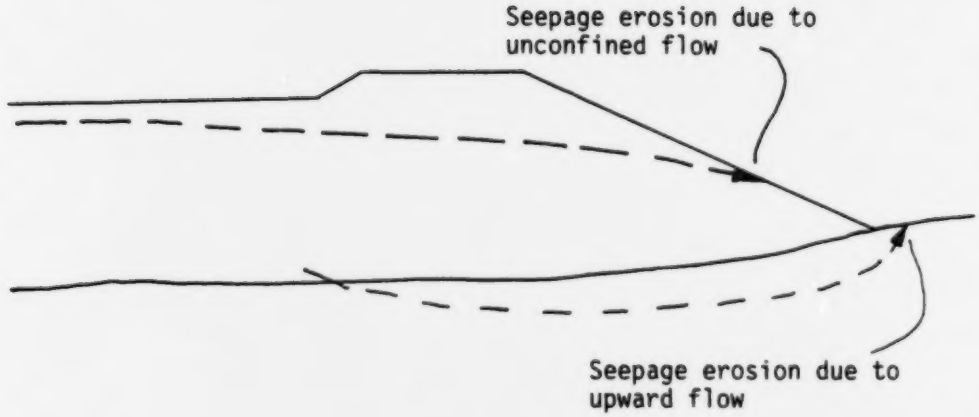


FIGURE III.8-1 SCHEMATIC OF POTENTIAL SEEPAGE EROSION FAILURES IN URANIUM TAILINGS IMPOUNDMENTS

TABLE III.8-1

CHARACTERIZATION OF SEEPAGE EROSION MODES
(Modified after van Zyl and Harr, 1981)

MODE	MECHANISM	METHOD OF ANALYSIS	EXAMPLES WITH RESPECT TO URANIUM TAILINGS IMPOUNDMENTS
Heave	. Simultaneous heaving of substantial soil volume . Shear resistance of soil volume approximately zero	. Comparison of seepage force per unit volume with effective unit weight of selected critical volume of soil	. will occur if gradient increases rapidly . can only occur due to upward flow
Piping	. Initiated locally at discontinuities in the soil, e.g., concentration of fines, particles with low specific gravity or fissures and cracks . Removal of particles begin at soil surface . A space is created at exit which progressively enlarges and works backwards to form irregular channels or "pipes" . Soil adjacent to pipes is stable	. Almost impossible due to control by discontinuities, however, use of "global" gradient approach (see below) is an attempt	. Can occur due to upward flow as well as unconfined flow. Both mechanisms are possible depending on on specific conditions.
Internal	. Begins locally by erosion of fine particles being moved from the soil matrix into a coarser layer . Cavities form, leading to collapse and failure	. Attempts have been made to combine effect of particle size distribution and gradient, generally very difficult to analyze. Prevent by using filter criteria	. This mode is considered of lower probability for uranium tailings impoundment

III.8.3.1 Global Gradient Approach

The principle of the global gradient approach to the design of weirs on permeable foundations was first stated by Buckley (1905). (Referred to by Khosla, et. al. (1936) as "Irrigation Works in India", no publisher is given). It was based upon his laboratory experiments in India during the last decade of the 19th century. Buckley concluded:

"It has been maintained that ... the true measure of security of a weir in a permeable bed is the distance through the soil which a current of water would have to travel before it could rise up below the weir, and it is of little consequence whether masonry is laid horizontally on the weir bed or sunk vertically below it, so long as currents passing through the soil below the structure are exposed to the friction of the same length of passage."

Studies based on the concept of the length of the path travelled by seeping water led to the development of the so-called creep ratios or creep coefficients. Two well-known design methods on this basis are those by Bligh (1927) (and earlier editions of this text) and Lane (1935). The method proposed by Lane (1935) is still widely used in the design of dams and weirs, despite criticism about its empirical nature. Table III.8-2 gives the definition of the creep coefficients in both cases as well as the suggested limiting magnitudes. In concept, creep coefficients obtained from the equations in Table III.8-2 are to be higher than those tabulated for the soil conditions under consideration. The limiting values suggested were obtained by analyzing a large number of structures founded on various soil conditions (Lane, 1935, analyzed 200 structures).

The global seepage gradient is defined as the head loss per unit length of flow path. The inverse of the creep coefficient is therefore the global seepage gradient. These values are also shown in Table III.8-2 ($1/C$ and $1/C_w$).

The limiting gradients or creep-coefficient in the global gradient approach were determined from an analysis of a large number of structures. The sample of structures investigated in each case may, however, be an insignificant part of the statistical population, and does not necessarily include all the possible failure modes or adverse subsoil conditions. In some cases where failure took place, it might have been caused by seepage along geological discontinuities (Terzaghi, 1929). Concentrated seepage, instead of "average seepage" therefore may be the cause (Davidenkoff, 1970). The prediction of the safety of a structure against seepage erosion caused by concentrated seepage is not possible with any of the global gradient techniques unless the source of the concentrated seepage is detected during site investigation or prevented during construction. Blind application of design methods based on global gradients can therefore lead to a false sense of security. However, as a first approach, these methods give a very good indication of potential problems.

An advantage of the global gradient approach is that the values of the suggested limiting gradients, or creep coefficients, are dependent on the subsoil conditions of the project site. It is therefore impressed upon the

TABLE III.8-2 CREEP COEFFICIENTS AND GLOBAL GRADIENT

	Bligh (1927)			Lane (1935)		
Expression	Creep Coefficient: $C = \frac{L}{h}$ where L = length of seepage path, measured along base of weir; h = total head loss			Weighted Creep Ratio: $C_w = \frac{H}{3} + V$ / h where H = distance along horizontal contacts ($<45^0$); V = distance along vertical contacts ($>45^0$); h = total head loss		
Limiting Values Suggested	Soil Type	C	1/C	Soil Type	C_w	$1/C_w$
	light silt and mud (60% passing 100 mesh sieve)	18	0.06	very fine sand or silt	8.5	0.12
	fine micaceous sand (80% passing 75 mesh)	15	0.07	fine sand	7.0	0.14
				medium sand	6.0	0.17
	coarse-grained sands	12	0.08	coarse sand	5.0	0.2
	mixture of boulders, gravel and sand	5-9	0.1-0.2	fine gravel	4.0	0.25
				medium gravel	3.5	0.29
				coarse gravel including cobbles	3.0	0.33
				boulders with some cobbles and gravel	2.5	0.4
				hard clay	1.8	0.56

designer that, e.g., silt and fine sand erode more easily than gravel or clay.

In applying the global seepage gradient approach to uranium tailings structures, it is recommended that the approach by Lane (1935) be adopted. The head loss through the foundation as well as the length of the seepage path must be estimated. It is expected that the dangers due this mode of failure will be low for uranium tailings impoundments.

Internal erosion in stratified natural deposits can influence the landscape and is usually referred to as "geologic piping" (Kalin, 1977). This process is suspected in the failure of many structures as described by Casagrande, et al (1972), Marsal and Pohlenz (1972) and Reuss, et al (1972). High flow velocities through highly permeable layers and discontinuities can lead to the removal of fine particles from the soil matrix and finally the formation of "pipes". These "pipes" may collapse, thereby causing failure of overlying structures.

III.8.3.2 Seepage Erosion Through Heave

Seepage erosion can be initiated through heave of material due to upward flow. This mechanism has been identified as a major source of seepage erosion immediately downstream from sheet-pile cutoffs (Terzaghi, 1943; Sentko, 1961). However, this failure mode can also be a source of seepage erosion downstream from uranium tailings impoundments. The most likely failure mechanism through heave can be related to previous field observations as described below.

Turnbull and Mansur (1961) presented measurement of upward gradients through the top stratum on the landward side of levees along the lower Mississippi River during the 1950 high water. Analyses showed that the magnitude of the gradient through the top stratum was associated with the different observed flow conditions given in Table III.8-3.

Domjan (1961) described the failure of levees along the Hungarian section of the Danube during the floods of 1954 and 1956. It was concluded that failures were initiated by the compression of air in the sand-gravel layer underlying a one meter to two meter thick relatively impermeable silt stratum. The air, which filled 20% to 30% of the void space of the sand-gravel layer, was compressed when the flood level rose and led to sudden expansion (comparable to an explosion) upon release through the top stratum. This sudden release initiated internal erosion and finally failure.

Kezdi (1976) called the process described by Domjan (1961) hydraulic failure. In three phase soils (solids, water, air) failure may take place at a gradient smaller than the critical gradient for saturated soils (two phases: solids and water). "The enclosed air bubbles are compressed during the process and thus positive neutral air-stresses come into being, thereby reducing the shear strength" (Kezdi, 1976).

The failures described above were due to a rapid rise in the seepage head. Since it is not expected that such a situation will develop at uranium

TABLE III.8-3

OBSERVED FLOW CONDITIONS ASSOCIATED WITH GRADIENT THROUGH TOP SOIL STRATUM
(Turnbull and Mansur, 1961)

Flow Condition	Gradient Through Top Stratum
Light to no seepage	0 to 0.5
Medium seepage	0.2 to 0.6
Heavy seepage	0.4 to 0.7
Sand boils	0.5 to 0.8

tailings impoundments, this failure mode therefore has a low probability of occurring.

III.8.3.3 Exit Gradient Approach to Seepage Erosion

In 1922, Terzaghi published his classic work on the failure of dams by seepage erosion. The theoretical development is based on the summation of the vertical seepage forces exerted by the upward flow of water and the vertical downward weight of the submerged soil particles, so that the critical gradient, i_c, is given by:

$$i_c = \frac{G - 1}{1 + e} = (1 - n)(G - 1) \qquad (8.1)$$

where G = specific gravity of soil particles
e = void ratio of soil
n = porosity of soil

For typical soils, assuming G = 2.65 and $0.28<n<0.52$, the value of i_c ranges from 0.8 to 1.2, with a mean of approximately 1.0 (Terzaghi, 1922).

The exit gradient (due to upward flow) for a specific set of boundary conditions can be obtained through approximate methods (such as flow nets) or through numerical methods.

In general, the factor of safety with respect to seepage erosion for the exit gradient approach is defined by:

$$FS = \frac{i_c}{i_e} \qquad (8.2)$$

Where i_c = critical gradient as defined in equation 8.1

i_e = exit gradient for specific boundary conditions

It is customary in geotechnical engineering to specify a minimum allowable factor of safety when a deterministic (design) approach is followed. The recommended factor of safety is typically a result of experience. Table III.8-4 gives a summary of different factors of safety against seepage erosion recommended by different authors. The large factors of safety with respect to seepage erosion reflect the degree of uncertainty in the knowledge of foundation conditions and the overall seepage erosion process. If the foundation material is homogeneous and the seepage boundary conditions can be defined with a high degree of certainty, lower values of the factor of safety can be used. This is reflected in the value of 1.5 recommended by Marsland (1953). This value was derived on the basis of carefully conducted laboratory experiments. The application of the exit gradient approach to uranium mill tailings is appropriate. However, it is recommended that high factors of safety (about 10) be used in this approach.

TABLE III.8-4

RECOMMENDED FACTORS OF SAFETY AGAINST SEEPAGE EROSION
(EXIT GRADIENT APPROACH)

Author	Soil Conditions	Factor of Safety
Casagrande (1935) (discussion of Lane's paper)	Homogeneous soil with anisotropy $k_{max}/k_{min} \geq 2$ to 3	3
	Poor site exploration or irregular and stratified deposits	≥ 10
Khosla, et al (1936)	Gravel	4 to 5
	Coarse Sand	5 to 6
	Fine Sand	6 to 7
Zaki and Leliavsky (1948)	Exit gradient from flownet. Allow for scouring downstream from weir.	15
Marsland (1953)	Variation of site conditions well established	1.5
Harr (1962)		≥ 4

III.8.3.4 Practical Considerations with Respect to Upward Flow

The results of the laboratory tests by van Zyl (1979) and the findings of other researchers (e.g. Terzaghi, 1922; Sentko, 1961) suggest that the following matters be given special consideration when concern centers on the seepage erosion of granular soils due to upward flow:

(i) The head at which seepage erosion failure occurs depends on the soil type. The amount of fines in the soil, the specific gravity of the constituent minerals and the porosity of the soil play the most important roles.

(ii) Discontinuities can lead to concentrated seepage ("piping") and increased probability of failure.

(iii) The mode of seepage erosion failure depends on the soil type, the rate of head increase and the flow condition (e.g. saturated or unsaturated flow). The soil type controls whether heave is followed by a quick condition as in clean sand, or whether heave leads to crack formation, concentrated flow and piping. The latter mode appears to be more prevalent in granular soils with a large percentage of fines.

The rate of head increase determines whether seepage erosion is manifested by a quick condition on the soil surface, or whether heave of the soil surface will occur. A quick condition prior to heave can be produced when the head is raised very slowly.

A rapid increase in head results in a heaving of the surface, leading eventually to a quick condition. Failures of this kind are expected downstream from a water retention structure being filled rapidly.

Field observations by Domjan (1961), Nakajima (1968) and Kezdi (1976) indicated that unsaturated soil fails at lower gradients than the critical gradient of the soil. The first filling of a reservoir may induce this type of failure.

III.8.4 SEEPAGE EROSION MECHANISMS AND ANALYSIS-UNCONFINED FLOW

Two approaches have been proposed to analyze seepage erosion due to unconfined flow: a global gradient approach and an exit gradient approach. The latter was mainly derived for the case of a downstream face with tailwater and is, for the most part, based on the critical exit gradient approach. Although this condition will seldomly be present at uranium tailings impoundments, the approach and the applicable equations will be presented.

III.8.4.1 Global Gradient Approach

Chugaev (1967) presents an approach for unconfined flow similar to that of Bligh (1927) and Lane (1935). The global gradient is defined as the head loss through the embankment divided by the average length of the seepage path taken from the pool to the drain, i.e. i=Z/L in Figure III.8-2.

After analyzing 161 homogeneous dams constructed in the U.S.S.R. and abroad, Chugaev (1967) proposed the limiting global gradients in Table III.8-5 for dams constructed of various materials.

It must be noted that no other methods have been proposed to analyze seepage erosion due to unconfined flow, i.e. a phreatic surface exiting the downstream embankment. The above approach is empirical and not necessarily generally applicable, especially when other than homogeneous sections are evaluated. However, the results may be used as a first approximation in the analysis of uranium tailings impoundments.

III.8.4.2 Exit Gradient Approach

For sloping exit faces Sharma and Rao (1969) and Veerabhadra Rao (1972) used summation of forces to find expressions for the critical exit gradients. The forces acting on an element of a sloping face are shown in Figure III.8-3. Two different modes of failure can be considered: failure by upward displacement of particles and failure by sliding or "slip" of the granular particles down the slope when the frictional resistance is overcome. In the former the streamlines are taken as normal to the slope (worst case operative at point A, Figure III.8-3) and failure is considered to take place when the seepage force is greater than the opposing force due to the submerged unit weight of the soil (an exit gradient approach). A sliding failure takes place if the frictional resistance of the submerged soil, parallel to the slope, is reduced sufficiently (as a result of the component of the exit gradient normal to the face). Expressions for the critical gradient for different boundary conditions are given in Table III.8-6. A comparison of these expressions shows that sliding will occur at a smaller gradient than the upward displacement (e.g., equations B and E in Table III.8-6). Sliding will take place until the sloping bed has readjusted itself to an equilibrium condition.

TABLE III.8-5

LIMITING GLOBAL GRADIENTS FOR UNCONFINED FLOW
(Chugaev, 1967)

Soil Type	(i) max for dams in service
Clay-concrete	1.00
Clay	1.00
Loess	0.85
Loam	0.70
Non-uniform sand	0.50
Sandy loam	0.40
Fine sands	0.35

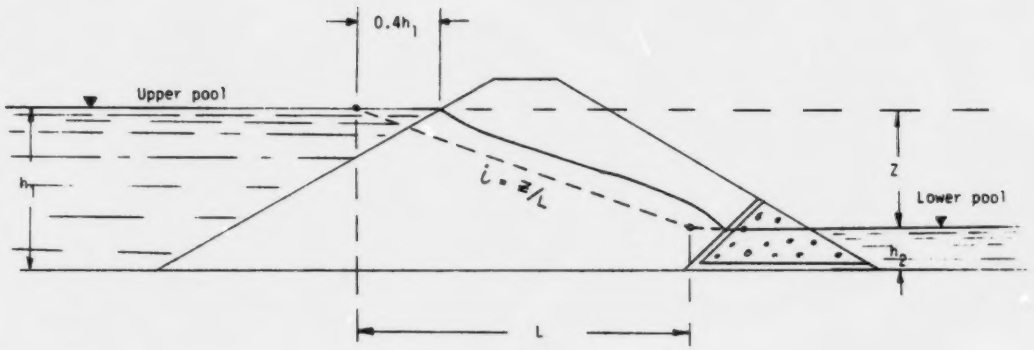
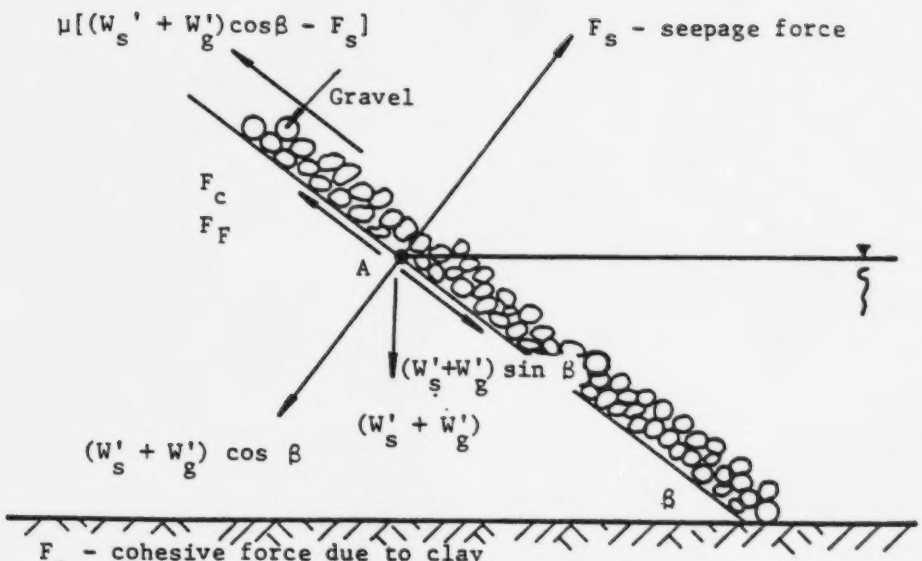


FIGURE III.8-2 DEFINITION OF GLOBAL GRADIENT (i=Z/L) FOR UNCONFINED FLOW (CHUGAEV, 1967)



F_c - cohesive force due to clay
F_F - frictional resistance between sand layers

$F_c = A_1\sigma_t$; $F_F = F_f A_2$; $\mu = \tan\phi$

$W_s' = (G_s - 1)(1 - n_s)\gamma_w V_s$

$W_g' = (G_g - 1)(1 - n_g)\gamma_w V_g$

FIGURE III.8-3 SEEPAGE EROSION OF SLOPING SURFACE (SHARMA & RAO, 1969)

TABLE III.8-6

EXPRESSIONS FOR CRITICAL EXIT GRADIENTS ON SLOPING EXIT SURFACES
(Sharma and Rao, 1969; Veerabhadra Rao, 1972)

---

1. Generalized solution of critical gradient for sliding failure. Gravel drainage layer on sloping face. Material is a mixture of cohesive and cohesionless soil.

$$I_{cs} = [(G_s - 1)(1 - n_s) + (G_g - 1)(1 - n_g)\frac{V_g}{V_s}] \frac{\sin(\phi - \beta)}{\sin \phi} + \frac{\sigma_t A_1}{V_s \gamma_w \tan \phi} + \frac{F_f A_2}{V_s \gamma_w \tan\phi} \quad (A)$$

(i) For cohesionless soils, without gravel layer and lateral confinement,

$$V_g = 0,\ \sigma_t = 0,\ F_f = 0$$

$$I_{cs} = (G_s - 1)(1 - n_s) \frac{\sin(\phi - \beta)}{\sin \phi} \quad (B)$$

(ii) For cohesive soils without gravel layer

$$I_{cs} = (G_s - 1)(1 - n_s) \frac{\sin(\phi - \beta)}{\sin \phi} + \frac{\sigma_t A_1}{V_s \gamma_w \tan \phi} + \frac{F_f A_2}{V_s \gamma_w \tan \phi} \quad (C)$$

2. Generalized solution of critical gradient for upward displacement of particles. Gravel drainage layer on sloping face. Material is a mixture of cohesive and cohesionless soil.

$$I_{cp} = [(G_s - 1)(1 - n_s) + (G_g - 1)(1 - n_g)\frac{V_g}{V_s}] \cos \beta + \frac{\sigma_t A_1}{\gamma_w V_s} + \frac{F_f A_2}{\gamma_w V_s} \quad (D)$$

(i) For cohesionless soils without gravel layer and lateral confinement

$$I_{cp} = (G_s - 1)(1 - n_s) \cos \beta \quad (E)$$

(ii) For cohesive soils, without gravel layer but with lateral confinement

$$I_{cp} = (G_s - 1)(1 - n_s)\cos\beta + \frac{\sigma_t A_1}{\gamma_w V_s} + \frac{F_f A_2}{\gamma_w V_s} \qquad (F)$$

where

A_1 - projected area of soil mass on a plane normal to direction of driving force

A_2 - contact area between sand layers

F_f - average frictional resistance per unit contact area

G_s - specific gravity of sand

G_g - specific gravity of gravel

n_s - porosity of sand

n_g - porosity of gravel

V_s - volume of sand mass

V_g - volume of gravel mass

β - angle of sloping surface with the horizontal

ϕ - angle of internal friction of sand

σ_t - tensile strength of soil

γ_w - unit weight of water

III.9.0 PROBABILITY METHODS IN FAILURE COMBINATIONS

III.9.1 PROBABILITY METHODS IN FAILURE EVALUATION

Most engineering designs are presently based on deterministic analyses. That is, single values are selected for material parameters and compared against the loads applied. In other words, the capacity of the system is compared to the demand placed on it, and this comparison is frequently expressed as a factor of safety. Such an approach neglects the importance of the variability which is inherent in all parameter values. For example, even a manmade material such as steel, which is subjected to stringent quality control, shows variability in yield tensile strength. Geological materials and processes shows much greater variability and to neglect such variability in any engineering design is unreasonable.

A probabilistic analysis considers the variability or uncertainty of the parameters. The central tendency of a parameter is usually expressed by the arithmetic mean while the coefficient of variation is a useful measure of parameter variability or dispersion. The coefficient of variation is the ratio of the standard deviation to the mean and is expressed as a percentage.

The final result of a probabilistic analysis is expressed as a reliability or probability of failure. This linearly scaled parameter is a much more realistic measure upon which to base decisions than a "factor of safety". The latter is not linearly scaled and its "accepted" value is based on experience.

Probabilistic analyses are not a replacement for engineering judgement. On the contrary, considerable judgement and knowledge of a process or failure mode is required to perform a realistic probabilistic analysis. The biggest advantage of a probabilistic analysis is that it forces the engineer to investigate the variability and uncertainty of all the contributing forces or parameters to a specific failure mode. Even when a precise quantification of probability of failure is not possible, systematic formulation of an analysis aids greatly in understanding the major sources of risk (Whitman, 1984).

Failure of tailings impoundments can take different forms. For example, some erosion could occur and remove a small amount of the toe of an embankment with no release of tailings. On the other hand, a massive loss of a large part of the impoundment could occur releasing large volumes of tailings over a large area. Obviously these two failures would have greatly different consequences. It is necessary, therefore, to consider not only probability of failure but the consequences of this failure as well. In this regard, the concept of "risk" and "hazard" should be introduced.

Risk may be defined as a compound measure of the probability and magnitude of adverse effects, or

Risk = Uncertainty X Damage

Other definitions of risk are "the chance of encountering harm or loss" or the "degree of probability of such loss" (Federal Emergency Management Agency, 1984).

The dictionary defines hazard as "a source of danger". Hazard, therefore, simply exists as a source. Risk includes the likelihood of conversion of that source into actual delivery of loss, injury or some form of danger, or

Risk = Hazard/Safeguards

This implies that risk may be kept as small as desired by increasing the safeguards. As a matter of practical reality, however, risk can never be brought to zero (Federal Emergency Management Agency, 1984).

Hazard is the possibility that some adverse effect might happen upon exposure. Risk is the probability that hazard will happen.

Harr (1984) and Whitman (1984) discuss various aspects of reliability analyses. The major principles and procedures, as they relate to uranium tailings impoundments are described below.

The variability/uncertainty in the value of any parameter x is expressed by a probability density distribution, as shown in Figure III.9-1. The probability that the parameter x will have values less than x = a is given by the area under the curve shown shaded in Figure III.9-1. The parameter x can represent the capacity of the structure to withstand load, e.g., the differential settlement which a cover material can withstand before severe cracking will occur. For example, the shaded area will be the probability that cracking will take place when the differential settlement (or demand on the structure) is a maximum of x = a. Therefore, if the demand is considered to be a deterministic value such as x = a in Figure III.9-1 and the capacity is assumed to have some distribution, then the shaded area is the probability of failure.

The demand on the structure, e.g. the predicted differential settlement, can also be a variable such as x in Figure III.9-1. In this case, one can consider a "capacity-demand model" as shown in Figure III.9-2. The probability of failure is a function of the area of overlap (note the probability of failure is not equal to the area of overlap).

As a comparison, note that the central factor of safety is defined as:

$$CFS = \bar{C} / \bar{D} \qquad (9.1)$$

Instead of using "all the information" contained in the probability distributions the factor of safety approach only uses two deterministic values, in the case of the central factor of safety the mean values.

There is one more approach to calculate the probability of failure on the basis of factor of safety. Consider the distribution in Figure III.9-1. to be the distribution of factor of safety. The probability of failure will then be given by the shaded area if a = 1, i.e., the probability of failure is the probability that the factor of safety is less than unity.

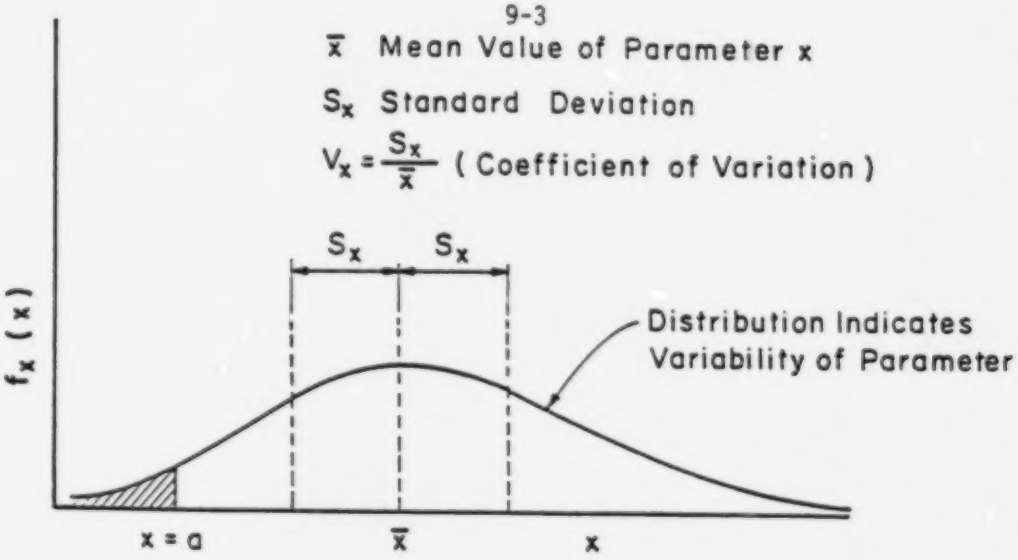


FIGURE III.9-1 PROBABILITY DENSITY DISTRIBUTION OF PARAMETER X

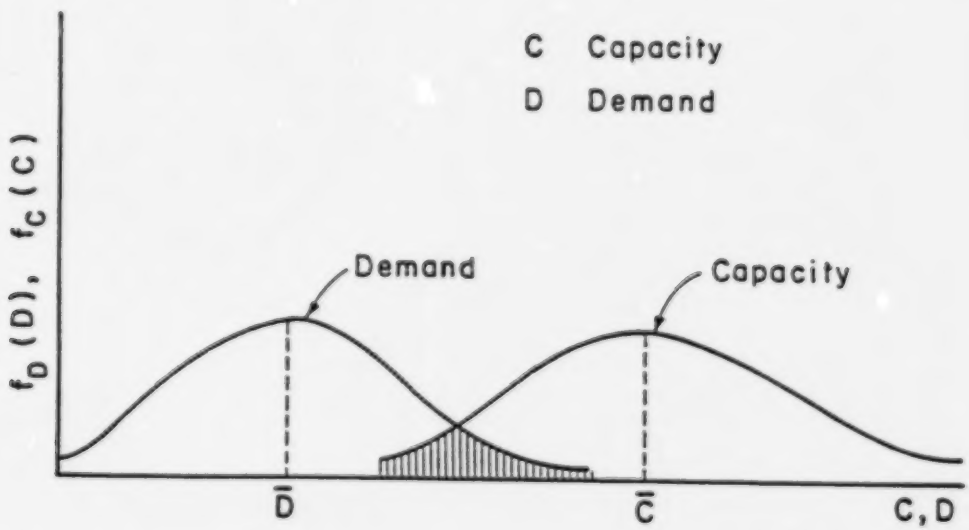


FIGURE III.9-2 CAPACITY-DEMAND MODEL

There are several requirements for the formal treatment of reliability (Whitman, 1984):

(i) Clear delineation of the criteria for success or failure.

(ii) Selection of a deterministic model relating the basic variables to the criteria for success or failure.

(iii) Identification of the uncertainties concerning the basic variables.

(iv) Evaluation of the distribution functions or moments of the basic variables.

III.9.1.1 Evaluation of Single Failure Modes

The probability of failure of single failure modes, e.g., slope stability, can be evaluated by applying the available deterministic models or by using them in conjunction with specially developed probabilistic models. The former approach is most often used, and its principles will be described.

As a first step in the analysis, the parameters whose variability will have a significant impact on the failure must be identified. In the case of slope stability, the shear strength parameters of the soil are the major variables influencing failure; pore pressure should also be considered as a major variable in some cases. Variability in the unit weight of the soil can be neglected in most cases.

The next task is to investigate and evaluate the variability of the significant parameters. This step may require considerable judgement when insufficient data are available. Some analysis techniques (as described below) require a knowledge of the probability density function of a parameter (such as Figure III.9-1.) while others only require the mean and standard deviation of the parameter to be known. A review of the literature has shown that most geotechnical parameters have quite consistent coefficients of variation. A summary of representative coefficients of variation for geotechnical parameters is given in Table III.9-1. These values may be used as a first approximation when specific test results are not available.

The selection of a probability distribution to represent geotechnical parameters is not an easy task. It has been found that the distribution of most geotechnical parameters can be approximated by a normal distribution (Lumb, 1966; Schultze, 1972). Hydraulic conductivity seems to follow a log normal distribution (van Zyl, 1979). Harr (1984) suggests the beta distribution as a more generalized distribution for all geotechnical parameters.

As discussed above, the probability of failure can be estimated by using the capacity-demand approach. It is first necessary to propagate parameter variability throughout the analysis so that the variability of the capacity and demand can be obtained. Three approaches are generally used for the propagation of uncertainty:

TABLE III.9-1

REPRESENTATIVE COEFFICIENTS OF VARIATIONS FOR GEOTECHNICAL PARAMETERS (HARR, 1984)

Parameter	Coefficient of Variation*,%
Porosity	10
Specific Gravity	2
Unit Weight	3
Hydraulic Conductivity	90 - 100
Compressibility	16
Compression Index	
. Sandy Clay	26
. Clay	30
Standard Penetration Test	26
Standard Cone	37
Friction Angle (Phi)	
. Gravel	7
. Sand	12
C-Parameter	40

*Coefficient of Variation = $\frac{\text{Standard Deviation}}{\text{Mean}}$ x 100%

(i) Monte Carlo method,
(ii) Taylor series approximation, and
(iii) Point-estimate method.

Complete knowledge of the probability distribution of each parameter is required for application of the Monte Carlo method. The approach is to generate random numbers for each parameter and to perform the calculations a large number of times in order to obtain the variability in the capacity and demand. Computer time requirements may become high if a large number of variables is present.

Only the mean and standard deviation of the variables are required to apply the Taylor series approximation and the point estimate method. The former requires that first order partial derivatives be formed of the function with respect to each of the variables. The point estimate method is an arithmetic approach. Both of these methods are described by Harr (1984).

The probability of failure for normally distributed capacity and demand is given by (Whitman, 1984):

$$P_f = 1 - \Phi(\beta) \qquad (9.2) \qquad \beta = \frac{\bar{C} - \bar{D}}{\sqrt{\sigma_C^2 + \sigma_D^2}} \qquad (9.3)$$

Where $\Phi(\beta)$ - cumulative standard normal distribution (obtained from a table)

$\bar{C}, \bar{D}$ - the mean values of capacity and demand

σ_C, σ_D - standard deviation of capacity and demand

If capacity and demand are both lognormally distributed:

$$\beta = \frac{\ln\left[\frac{\bar{C}}{\bar{D}}\sqrt{\frac{1+V_D^2}{1+V_C^2}}\right]}{\sqrt{\ln[(1+V_D^2)(1+V_C^2)]}} \qquad (9.4)$$

Where V_C, V_D - coefficients of variation of capacity and demand

When other probability distributions are used, convolution with numerical integration is used to obtain the probability of failure (Harr, 1984).

Complete probabilistic analyses of single failure modes will encompass all the steps above.

III.9.1.2 Evaluation of Combined Failure Modes

Combination of independent failure modes in a complex system can be done through the use of fault trees and event trees. A fault tree analysis may

include a quantitative evaluation of the probabilities of the various faults or failure events leading eventually to the calculation of the probability of the main failure event (top event). The potential consequences may be systematically identified with the event tree diagram in which all the possible paths following a top event or initiating event may be traced through the subsequent events, each path leading to a different consequence (Ang and Tang, 1984). Figure III.9-3 shows schematic fault and event tree diagrams.

A fault tree is the graphical development of a top event (major fault) into the union and/or intersection of subevents (faults). "OR" gates indicate the union of subevents, such as E being the union of E1 and E2, in Figure III.9-3. In other words, E will occur if either E1 or E2, (or both) occurs. The "AND" gate signifies that E2 is the intersection of subevents E3 and E4, implying that the occurrences of both subevents E3 and E4 are necessary for the occurrence of E2. When the probabilities are small "OR" gates can be considered as the sum of two probabilities, e.g., P1 = P5 + P6. For "AND" gates, the product of the probabilities is used, e.g., P2 = P3P4.

In an event tree, the particular consequence depends on the subsequent events following the initiating event. Given an initiating event there may be several "first subsequent events" that will follow, all of which must be mutually exclusive and exhaustive. Each path in the event tree represents a specific sequence of (subsequent) events, resulting in a particular consequence. The probability associated with the occurrence of a specific path is simply the product of the probabilities of all the events on that path (if the events are independent, an assumption usually made).

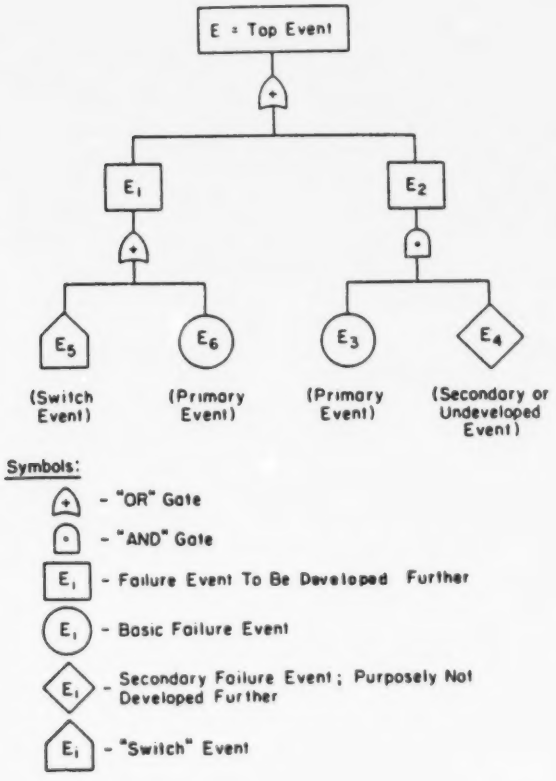
The probabilities used as input to fault trees and event trees can be calculated as discussed above for single failure modes, or by some other objective probabilistic approach. However, in many cases it is necessary to use subjective probabilities obtained through engineering judgement. The use of subjective probabilities does not reduce the rigor of the reliability analysis. It could be used as a basis for sensitivity analysis, where the subjective probabilities are varied over the most probable range and their influence on the result investigated.

III.9.2. EXAMPLES OF PROBABILISTIC ANALYSES

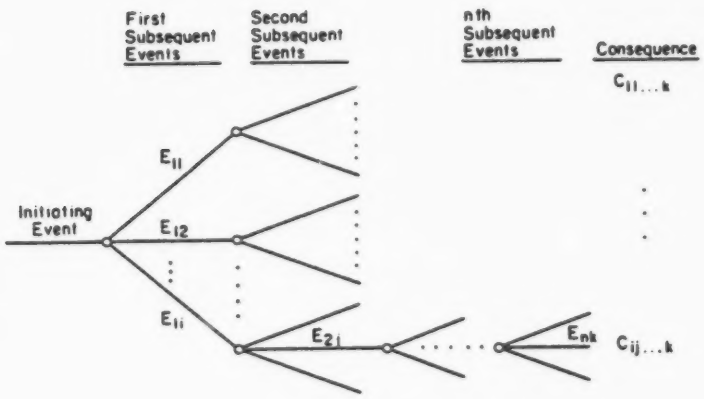
A number of examples of applicable probabilistic analyses for single failure modes (e.g., slope stability) and failure combinations is presented by Whitman (1984) and Ang and Tang (1984). Two examples of combined failure analyses will be given here to demonstrate the use of fault trees and event trees.

Example 1: Failure of Storage Facility (Excerpted by Ang and Tang, 1984, from Lambe, Marr and Francisco, 1981)

In the safety assessment of a dam designed for waste storage, the most serious event will consist of the loss of contaminated fluid stored within the waste storage facility. The following five failure mechanisms may result in the loss of fluid:



(a) Fault tree diagram.



(b) Event tree model.

FIGURE III.9-3 SCHEMATIC FAULT AND EVENT TREES (ANG & TANG, 1984)

(i) Excessive flow (i.e., exceeding the capacity of the perimeter collection ditch) through the dam as a result of highly permeable soil present in the dam and the poor performance of the cutoff system.

(ii) Overtopping caused by excessive rainfall. On the other hand, if the spillway is plugged (e.g., by vegetation and debris), overtopping could occur.

(iii) A sinkhole occurs within the area of the dam that subsequently expands leading to a loss of containment.

(iv) Shear slide occurs through the crest of the dam.

(v) Piping (or internal erosion) developed through the dam or foundation. Piping may start with small cracks that gradually widen, eventually leading to a concentrated flow of containment in the absence of a well-designed cutoff system. The fault tree representation of the loss of contaminated fluid is shown in Figure III.9-4.

Example 2: Evaluation of Geotechnical Aspects of Dam Safety (Whitman, 1984)

Figure III.9-5(d) presents a simple event tree, in which each branch point represents some characteristic of the dam that has a major effect upon safety. The words used to denote these events must be interpreted liberally, e.g. the phrase "dam cracks" would include any happening that might make the flow of water enough to begin internal erosion within the embankment, foundation or abutments. This simple event tree may be used to identify the important features that must be evaluated in order to characterize overall safety. It provides a systematic approach to account for multiple lines of defense against failure. For illustrative purposes, probabilities are given for factors affecting cracking of core, performance of filter and detection of piping in Figures III.9-5 (a to c). Finally, probabilities are assigned as shown in Figure III.9-5(d) to each branch to estimate the lifetime probabilities applicable for good conservative practice.

III.9.3 FAILURE MODES AND COMBINATIONS FOR URANIUM TAILINGS IMPOUNDMENTS

Nelson, et al (1983) identifies failure modes in the long-term stabilization of uranium tailings impoundments. These modes can be divided into two groups: those initiated by PMF (probable maximum flood) in the area adjacent to the impoundment and those associated with failure on the impoundment. A fault tree of the seven failure modes is presented in Figure III.9-6.

McInnis (1985) identifies the longterm potential failure modes listed in Table III.9-2. As is expected, considerable overlap exist between these and the failure modes presented in Figure III.9-6.

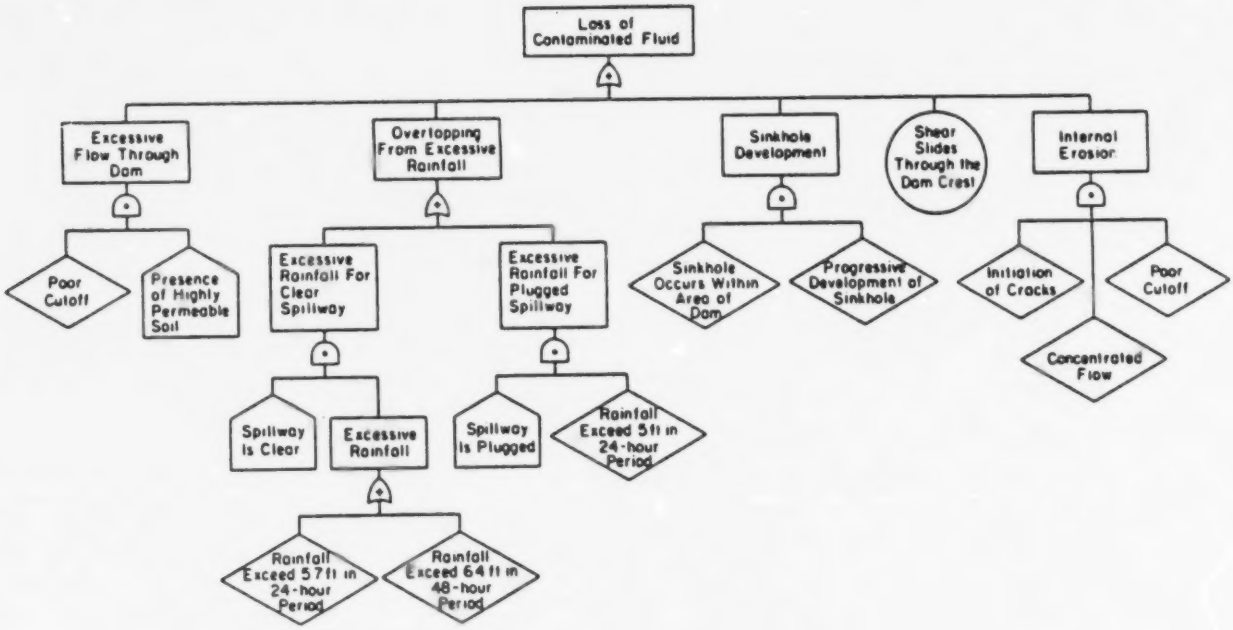
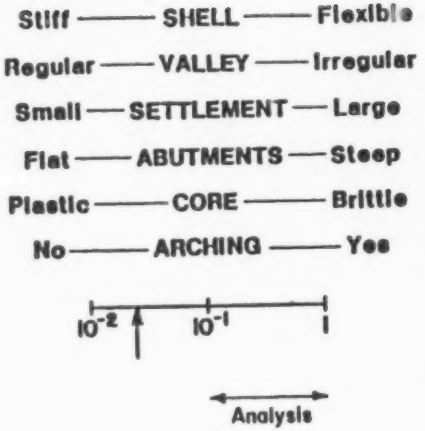
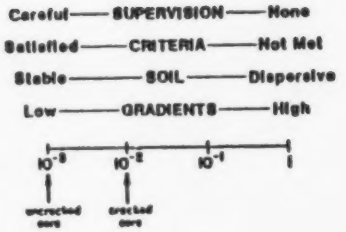

Loss of Contaminated Fluid
Excessive Flow Through Dam
Overtopping From Excessive Rainfall
Sinkhole Development
Shear Slides Through the Dam Crest
Internal Erosion
Poor Cutoff
Presence of Highly Permeable Soil
Excessive Rainfall For Clear Spillway
Excessive Rainfall For Plugged Spillway
Sinkhole Occurs Within Area of Dam
Progressive Development of Sinkhole
Initiation of Cracks
Poor Cutoff
Concentrated Flow
Spillway Is Clear
Excessive Rainfall
Spillway Is Plugged
Rainfall Exceed 5 ft in 24-hour Period
Rainfall Exceed 5.7 ft in 24-hour Period
Rainfall Exceed 6.4 ft in 48-hour Period

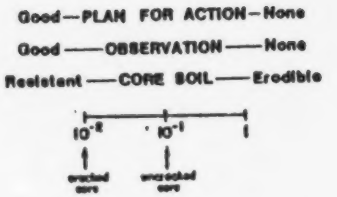
FIGURE III.9-4 FAULT TREE FOR EXAMPLE 1 OF LOSS OF CONTAMINATED FLUID



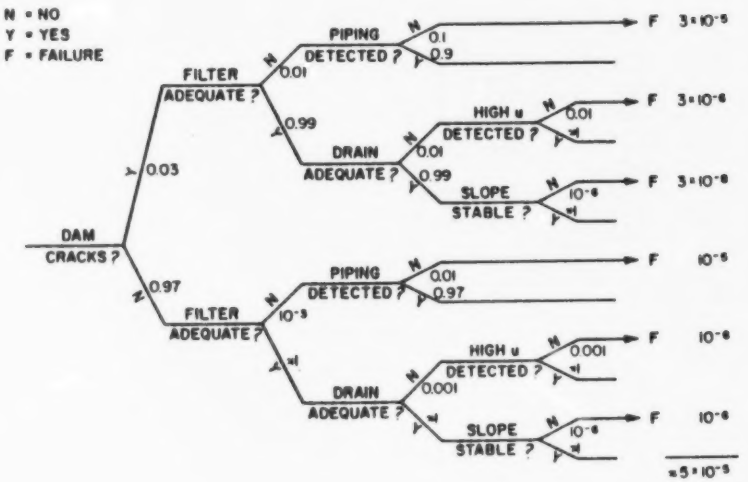
(a) Factors Affecting Cracking of Core



(b) Factors Affecting Performance of Filter



(c) Factors Affecting Detection of Piping



(d) Event Tree

EVENT TREE FOR EVALUATION OF GEOTECHNICAL ASPECTS OF DAM SAFETY AND THE FACTORS AFFECTING PERFORMANCE (WHITMAN, 1984)

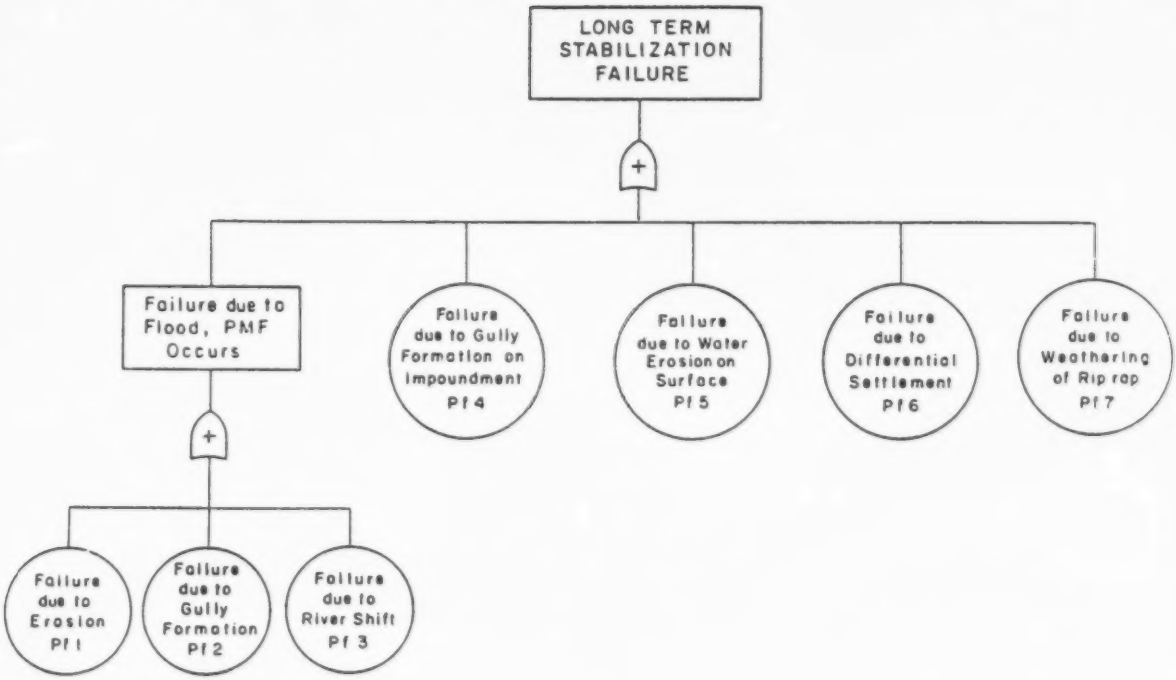

LONG TERM STABILIZATION FAILURE
+
Failure due to Flood, PMF Occurs
Failure due to Gully Formation on Impoundment Pf 4
Failure due to Water Erosion on Surface Pf 5
Failure due to Differential Settlement Pf 6
Failure due to Weathering of Rip rap Pf 7
+
Failure due to Erosion Pf 1
Failure due to Gully Formation Pf 2
Failure due to River Shift Pf 3

FIGURE III.9-6 FAULT TREE FOR LONG TERM STABILIZATION FAILURE, BASED ON FAILURE MODES IDENTIFIED BY NELSON, ET AL. (1983)

TABLE III.9-2

LONG TERM POTENTIAL MODES OF FAILURE
(MCINNIS, 1985)

Cover
- Differential settlement
- Gullying
- Sheet erosion
- Wind erosion
- Flooding
- Chemical Attack
- Shrinkage

Liners
- Differential Settlement
- Subsidence of Subsoil or Rock
- Chemical Attack

Embankments
- Slope Failure
- Differential Settlement
- Gullying
- Sheet Erosion
- Wind Erosion
- Flooding
- Weathering and Chemical Attack

Revegetation
- Fire
- Climatic Changes

Water Diversion Structures
- Slope Failure
- Obstruction

The present study has identified and classified the failure mechanisms as summarized in Table III.9-3.

The information contained in Figure III.9-6 and Tables III.9-2 and III.9-3 will next be considered to evaluate failure combinations and subsequently the total long term probability of failure of a uranium tailings impoundment. The approach followed here will result in an overall fault tree describing the failure mechanisms. It must be noted that similar results can be obtained by using the so-called 'parallel' and 'series' approaches to failure mechanisms (see e.g., Harr, 1977 and Frankel, 1984).

Failure is defined here as a surface release of uranium tailings resulting from a cover failure or an embankment failure. It is recognized that liner failures leading to groundwater contamination should also be considered in the overall environmental impact of uranium tailings impoundment. However, as the purpose of this study is to evaluate the long term stability of covers and control structures, liner failures will therefore not be considered.

III.9.3.1 Cover Failure

Cover failure can occur due to the modes listed in Table III.9-3 plus frost action. The effects of cover shrinkage are neglected in this analysis. A fault tree combining these failure modes is given in Figure III.9-7. It is assumed that excessive gully erosion, sheet erosion and wind erosion will directly result in dispersion of tailings. The other three modes: differential settlement, chemical attack and frost action, will result in disruption of the cover but flooding is required to disperse the tailings. The 'flooding' event will not necessarily have the same probability for all three modes of failure, e.g., a smaller flood may cause dispersion of tailings if considered in combination with differential settlement than if in combination with frost action.

The fault tree in Figure III.9-7 neglects the influence of man and animal intrusion. It is further clear that some failure modes could be further developed but this was not done, e.g., gullying which is a function of the steepness of the slope, drainage area, soil type, etc.

III.9.3.2 Control Structure (Embankment) Failure

The major failure modes for control structures are: slope failure, gullying, sheet erosion, flooding, frost action. Fault trees for those failure modes and their combination are shown in Figures III.9-8 and III.9-9. Further development of some failure modes may be possible, but was not done in order to maintain clarity.

The fault tree for the overall long term stability of a uranium tailings impoundment is given in Figure III.9-10.

III.9.3.3 Evaluation of Probabilities

The probabilities of the basic failure events (shown in circles) must be evaluated in order to obtain the overall failure probability. As mentioned above, objective or subjective probabilities can be used. It is further

TABLE III.9-3

FAILURE MECHANISM CLASSIFICATION OF THIS STUDY

A. Surface Mechanisms

(a) Erosion

(i) Sheet erosion
(ii) Rill erosion
(iii) Flood erosion

(b) Biological Effects

(i) Intrusion by man and animals
(ii) Negative effects of vegetation, e.g., perforation of low permeability covers and clogging of coarse suction barriers
(iii) Bacterial action causing long term drain plugging

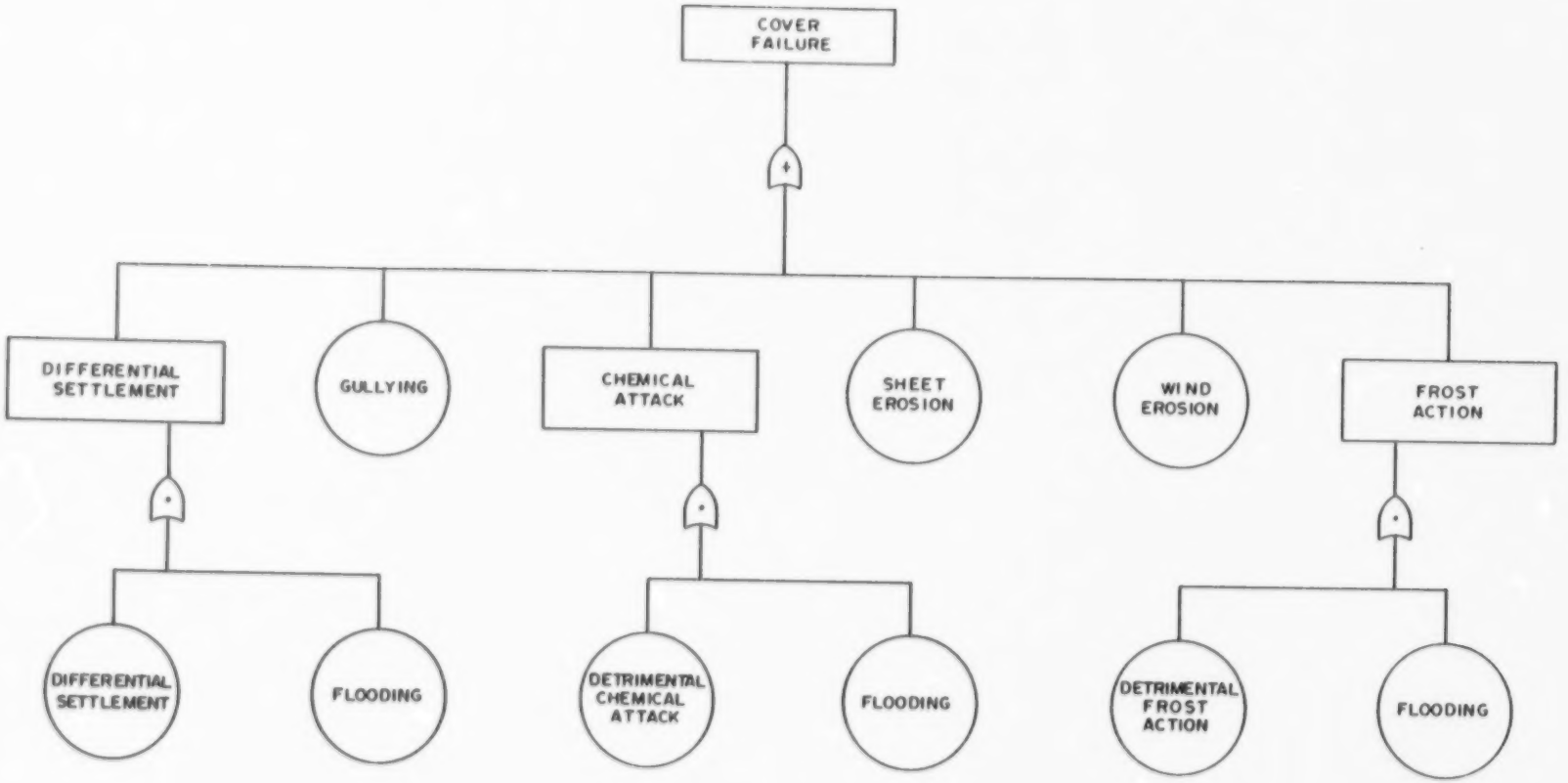
B. Sub-Surface Mechanisms

(a) Frost action such as seasonal freezing and intermittent permafrost

(b) Increase in height of phreatic surface

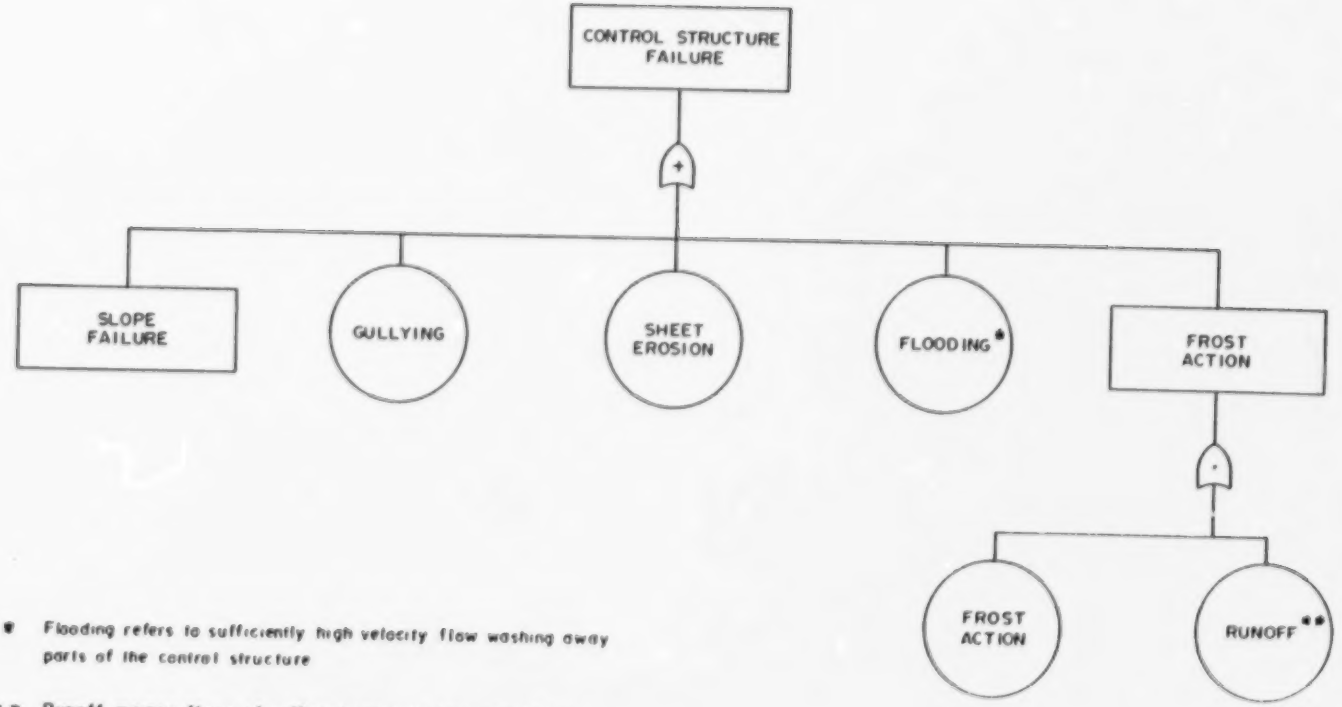
(c) Plugging of underdrains

(d) Material degradation, e.g., rip-rap



FAULT TREE FOR COVER FAILURE

FIGURE III.9-7



* Flooding refers to sufficiently high velocity flow washing away parts of the control structure

** Runoff means flows of sufficient intensity to cause failure after disruption by frost action

FAULT TREE FOR CONTROL STRUCTURE (EMBANKMENT) FAILURE

FIGURE III.9-8

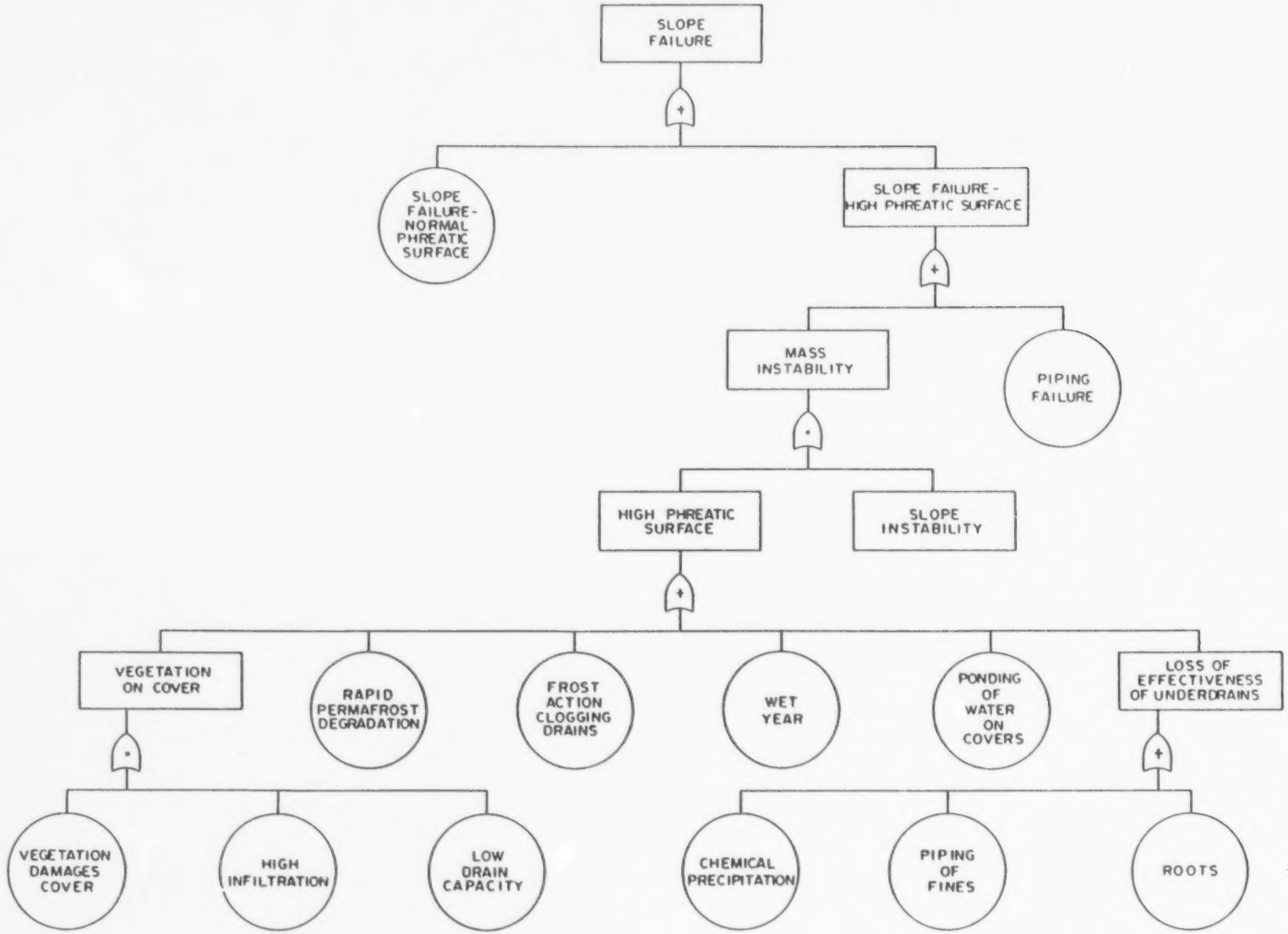


FIGURE III.9-6

FAULT TREE FOR SLOPE FAILURE OF CONTROL STRUCTURE (EMBANKMENT)

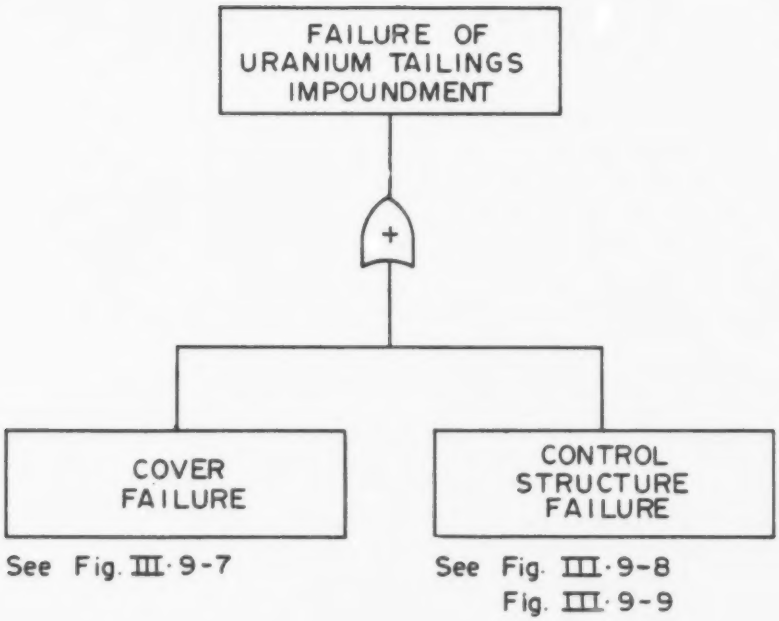


FIGURE III.9-10 FAULT TREE FOR OVERALL LONGTERM STABILITY OF URANIUM TAILINGS IMPOUNDMENT

important to select the 'time' at which the evaluation will be done. For example, there is a higher probability that vegetation will damage the cover or that the underdrains will clog 200 years after reclamation than 1 year after reclamation. The analysis can therefore be repeated for different times or a hazard function approach can be used (Frankel, 1984).

III.9.4 INTERPRETATION OF RESULTS

The results of a deterministic analysis (usually a factor of safety) are compared to allowable factors of safety obtained on the basis of experience. The results of a probabilistic analysis can be used in the same way, if an 'acceptable' probability of failure can be obtained. This is not necessarily a simple task. The probabilistic approach is well suited for the evaluation of alternatives, based on economic or other constraints.

A more appropriate approach would be to perform a probabilistic risk assessment once the failure probabilities are available (Bamberg and Van Zyl, 1986). In such an analysis the human health and environmental consequences of a failure must be evaluated considering the populations at risk. Such a full scale risk assessment can help to evaluate the cost/benefits of alternative designs more rigorously.

III.10.0 REFERENCES

American Geological Institute, Glossary of Geology, 1974. Washington, D.C.

American Society of Civil Engineers and the Water Pollution Control Federation, (ASCE) 1970, "Design and Construction of Sanitary and Storm Sewers", ASCE, 345 E. 47th St., New York, NY 10017.

American Society of Civil Engineers (ASCE) 1975. Sedimentation Engineering. American Society of Civil Engineering, New York, NY. 745 pp.

American Society of Civil Engineers (ASCE) 1977. Sedimentation Engineering. ed. Vanoni, ASCE Headquarters, New York.

Amerman, C.R. 1973. Hydrology and Soil Science in Field Soil Water Regime, Edited by R.R. Bruce, et al. Special Publication No. 5, Soil. Sci. Soc. Amer., Madison, Wisconsin, 1973.

Andersland, O.R. and Anderson, D.M., 1978. Geotechnical Engineering for Cold Regions. McGraw Hill, New York.

Ang, A.H.S. and Tang, W.H. 1984, Probability Concepts in Engineering Planning and Design, Vol. II: Decision, Risk and Reliability, John Wiley & Sons, Inc., New York, 562 p.

Anon, 1980. Trees for Reclamation. A Symposium sponsored by the Interstate Mining Compact Commission and U.S.D.A. Forest Service. 135 p. Northeast Forest Experiment Station, Broomall, P.A. U.S.D.A. For. Serv. Gen. Tech. Rep. NE-61.

Armbrust, D.V. and Dickenson, J.D., 1971. Temporary Wind Erosion Control: Cost and Effectiveness of 34 Commercial Materials. J. Soil and Water Conservation 26: 154-157.

Armson, K.A. 1977. Forest Soils; Properties and Processes, University of Toronto Press, Toronto.

Armson, K.A. and Williams, J.R. N., 1960. The Root Development of Red Pine (Pinus resinosa Ait.) Seedlings in Relation to Various Soil Conditions, For. Chron, 36; 14-17.

Aron, G., Miller, A.C., and Lakato, D.F. 1977. Infiltration Formula Based on SCS Curve Number, Journal of the Irrigation and Drainage Division 13427-IR4.

Asseed, M. and Swartzendruber, D., 1974. "Water Infiltration into Uniform and Stratified Soils. I. Review and Use of an Approximate Theory," Libyan J. Agric., Vol. 3, pp. 19-26.

Ayars, J.E. 1976. Salt Transport in Irrigated Soils. Ph.D. Thesis, Department of Agricultural Engineering, Colorado State University, Fort Collins, Colorado.

Babu, D.K. 1976a. Infiltration Analysis and Perturbation Methods, 1A, Sorption with Exponential Diffusivity, Water Resource Research 12, 80-9.

Babu, D.K. 1976b. Infiltration Analysis and Perturbation Methods, 2, Horizontal Absorption, Water Resource Research 12.

Ball, J.W., Jenne, E.A. and Nordstrom, D.K. 1979. "WATEQ2: A Computerized Chemical Model for Trace and Major Element Speciation and Mineral Equilibria of Natural Waters." In Chemical Modelling in Aqueous Systems, ed. E.A. Jenne, pp. 815-835. Amer. Chem. Soc. Symp. Series 93.

Ball, J.W., Jenne, E.A. and Cantrell, M.W., 1981. WATEQ3: A Geochemical Model with Uranium Added. U.S. Geol. Survey, Open File Report 81-1183.

Bamberg, S.A. and van Zyl, D., 1986. Probabilistic Risk Analysis in Waste Disposal - An Integrated Approach, Proc. of 8th Geotechnical & Geohydrological Aspects of Waste Management, A.A. Balkema Publishers, pp. 399-406.

Barfield, B.J., Hayes, J.C., and Barnhisel, R.I., 1978. The Use of Grass Filters for Sediment Control in Strip Mine Drainage. IMMR 39-RR4-78, Institute for Mining and Minerals Research, Lexington, Kentucky.

Barfield, B.J., Warner, R.C., and Haan, C.T. 1981. Applied Hydrology and Sedimentology for Disturbed Areas, Oklahoma Technical Press, Stillwater, Oklahoma.

Bayda, E.D., Groome, A.J., and McCallum, K.J. 1978. The Cluff Lake Board of Inquiry Final Report, Environment Saskatchewan, Regina, Canada.

Bear, J. 1972. Dynamics of Fluids in Porous Media. American Elsevier, New York.

Beedlow, P.A. 1984. Designing Vegetative Covers for Long-Term Stabilization of Uranium Mill Tailings. Report PNL-4986/NUREG/Cr 3674, Pacific Northwest Laboratories, U.S. Nuclear Regulatory Commission, Washington, D.C.

Beedlow, P.A. and Parker, G.B. 1985. Designing Protective Covers for Uranium Mill Tailings Piles: A Review. Report PNL-5323/NUREG/CR-4075, Pacific Northwest Laboratories, U.S. Nuclear Regulatory Commission, Washington, D.C.

Beedlow, P.A. and Hartley, J.M. 1984. Long-Term Protection of Uranium Mill Tailings, Pacific Northwest Laboratory Report DOE/UMT-0218.

Begin, Z.B. and Schumm, S.A. 1979. Instability of Alluvial Valley Floors: A Method for Its Assessment. ASAE Transactions, Vol. 22, pp. 347-350.

Berg, W.A. 1978. Limitations on the Use of Soil Tests on Drastically Disturbed Lands in: "Reclamation of Drastically Disturbed Lands" Edited by F.W. Schaller and P. Sutton, American Soc. of Agronomy, Madison, Wisconsin, U.S.A.

Bertrand, A.R. and Parr, J.F. 1971. Design and Operation of the Purdue Sprinkler Infiltrometer. Purdue University, Indiana, Agric. Exp. Stat. Bull. 723.

Betson, R.P. 1964. What is Watershed Runoff? J. Geophys. Res. 69:1541-1552.

Betson, R.P. 1972. Upper Bear Creek Experimental Project - A Continuous Daily-Stream Flow Model, TVA, Knoxville, Tennessee, February.

Blench, T. 1961. Sediment Transport in Open Channels: Hydraulics of Canals and Rivers of Mobile Boundary, in "Butterworth's Civil Engineering Reference Book", 2nd ed., Butterworth, London.

Bligh, W.G. 1927. The Practical Design of Irrigation Works, Van Nostrand Co., New York, 425 pp.

Bloomsburg, C.L. and Corey, A.T., 1964. Diffusion of Entrapped Air from Porous Media. Hydrology Paper No. 5 Colorado State University, Ft. Collins, Colorado.

Bouwer, H. 1966. Rapid Measurement of Air Entry Value and Hydraulic Conductivity of Soil as Significant Parameters in Flow System Analysis. Water Resources Research 2:4:729-738.

Bouwer, H. 1969. Infiltration of Water into Non-uniform Soil. Proc. ASCE 95 IR4:451-462.

Bouwer, H. 1976. Infiltration into Increasingly Permeable Soils. Proc. ASCE 102:IRI:127-136.

Bradshaw, A.D. and Chadwick, M.J. 1980. Restoration of Land. Studies in Ecology, Volume 6, Blackwell Scientific Publications, Oxford, U.K.

Brady, N.C. 1974. The Nature and Properties of Soils, MacMillan Publ. Co. Inc., New York, N.Y.

Brakensiek, D.L. and Onstad, C. 1977. Parameter Estimation of the Green and Ampt Infiltration Equation. Water Resour. Res. 13(6):1009-1012.

Brakensiek, D.L., Rawls, W.J., and Hamon, W.R. 1977. Application of an Infiltrometer System for Describing Infiltration into Soils. Paper No. 77-2553 presented at the 1977 Winter Meeting ASAE, Chicago, Illinois, 1977.

Brooks, B.W. 1981. Revegetation of the Waite Amulet Tailings Discharge Area. Proceedings of the 13th Annual Meeting of Canadian Mineral Processors, Ottawa, pages 341-363. Canadian Institute of Mining and Metallurgy, Montreal, P.Q.

Brooks, R.H. and Corey, A.T., 1964. Hydraulic Properties of Porous Media. Hydrology Paper No. 3, Colorado State University, Fort Collins.

Brown, B.L, 1982. Reclamation and Topsoil Use. American Mining Congress Journal, June: 48-54.

Brutsaert, W.F. 1971. A Functional Iteration Technique for Solving the Richard's Equation Applied to Two-Dimensional Infiltration Problems. Water Resources Research 7:61583-1596.

Bubenzer, G.D. and Jones, B.A., Jr. 1971. Drop Size and Impact Velocity Effects on the Detachment of Soils under Simulated Rainfall. Trans. ASAE 14:4:625-628.

Burns, S.F., 1980. Alpine Soil Factors in Disturbance and Revegetation. Proceedings of High Altitude Revegetation Workshop No.4. pages 210-227, Information Series No. 42, Colorado State County, Fort Collins, Colorado, U.S.A.

Carslaw, H.S. and Jaeger, J.C., 1959. Conduction of Heat in Solids, Oxford University Press, London, England.

Caruccio, F.T., 1978. The Nature of Acid Mine Drainage Reactions In: M.K. Wali (Editor) Ecology and Coal Resource Development Volume 2, pp 775-781, Pergamon Press, New York, N.Y.

Casagrande, A., Wilson, S.D. and Schwantes, E.D. (1972) The Baldwin Hills Reservoir Failure in Retrospect, Proc. of Speciality Conf. on Performance of Earth and Earth-Supported Structures, ASCE, Purdue University, June 11-14, Vol. 1, Part 1, pp. 551-588.

Chamber of Mines of South Africa, 1979. Handbook of Guidelines for Environmental Protection. Volume 2. The Vegetation of Residue Deposits Against Wind and Water Erosion. Chamber of Mines of South Africa, Johannesburg, R.S.A.

Cherry, J.A., Blackport, R.J., Dubrovsky, N., Gillham, R.W., Lim, T.P., Murray, D., Reardon, E.J., and Smyth, D.J.A. 1980. Subsurface Hydrology and Geochemical Evolution of Inactive Pyritic Tailings in the Elliot Lake Uranium District, Canada. Proceedings, Third Symposium on Uranium Tailings Management, Colorado State University, Ft. Collins, Colorado, No. 24-25, Session 5.

Cherry, J.A., Shepherd, T.A., and Morin, K.A. 1982. Chemical Composition and Geochemical Behavior of Contaminated Groundwater at Uranium Tailings Impoundments. SME-AIME Annual Meeting, Dallas, Texas, February 14-18.

Chepil, W.S., and Woodruff, N.P. 1963. The Physics of Wind Erosion and Its Control. Adv. Agron. Vol. 15, pp. 211-302.

Childs, E.C. 1969. An Introduction to the Physical Basis of Soil Water Phenomena. John Wiley & Sons, New York, 495 p.

Childs, E.C. and Bybordi, M. 1969. The Vertical Movement of Water in Stratified Porous Material. 1. Infiltration. Water Resources Research 5:2:446-459.

Chow, V.T., 1959. Open-Channel Hydraulics, McGraw-Hill Book Company.

Chu, C.S. and Bowers, C.E. 1977. Computer Programs in Water Resources. WRRC, University of Minnesota Graduate School Bull. No. 97.

Chugaev, R.R. 1967. Resistance of Earth Dams to Random Seepage, Hydrotechnical Construction (Translated from the Russian of Gidrotekhnicheskoe Stroitel'stvo), No. 2, pp. 132-134.

Clapp, R.B. and Hornberger, G.M. 1978. Empirical Equations for some Soil Hydraulic Properties. Water Resources Research Vol. 14, No. 14.

Clifton Associates Ltd., 1986. Evaluation of Natural Materials as Long Term Barriers for Uranium Mine Tailings - Phase I. DSS-NUTP Report #15SQ.23241-4-1696. March.

Collins, R.E. 1961. Flow of Fluids Through Porous Materials. Reinhold, N.Y.

Corey, G.L., Corey, A.T. and Brooks, R.H. 1965. Similitude for Nonsteady Drainage of Partially Saturated Soils. Hydrology Paper 9, Colorado State University, Ft. Collins, Colorado.

Crosby, E.C. and Overton, D.E. 1979. A Storm Soil Loss Model Derived form Erosion and Runoff Mechanics. Hydraulic Transport Modelling Symposium, Sponsored ASAE, New Orleans.

Davidenkoff, R. 1970. Unterlaufkigkeit von Stauwerken, Werner-Verlag, Dusseldorf, 103 pp.

Dave, N.K., and Lim, T.P. 1982. Hydrogeochemical Evolution of an Inactive Pyritic Uranium Tailings Basin and Retardation of Contaminant Migration in a Surrounding Aquifer. International Symposium on Management of Wastes from Uranium Mining and Milling, Albuquerque, New Mexico, USA, 10-14 May.

Dave, N.K., Lim, T.P., and Cloutier, N.R. 1985. Ra-226 Concentrations in Blueberries Vaccinium Angulstifolium Ait. Near an Inactive Uranium Tailings Site in Elliot Lake, Ontario, Canada. Environmental Pollution (Series B) 10, pp. 301-314.

Dean, K.C., Havens, R. and Glantz, M.W., 1974. Methods and Costs for Stabilizing Fine-Sized Mineral Waste. Bureau of Mines Report of Investigations 7896. U.S. Department of the Interior, Washington, D.C.

Dixon, R.M. 1978. Water Infiltration Control in Rangelands: Principles and Practices. Southwest Rangeland Watershed Research Center USDA-ARS, Tucson, Arizona, USA. For publication in the Proceeding of the First International Rangeland Congress Denver, Colorado, USA 14-18 August.

Domjan, J. 1961. Piping of Soils Near Dams, Proc. of the 5th Int. Conf. on SM & FE, Vol. 2, pp. 581-583.

DSMA-ATCON LTD., 1985. Report 1357/1335 A Study to Measure and Evaluate the Effect that Vegetation has on the Emanation Rate of Radon from a Uranium Tailings Pile, prepared for Canada DSS for National Uranium Tailing Program.

Dubrovsky, N.M., Cherry, J.A., Reardon, E.J. and Vivyurka, A.J., 1985. Geochemical Evolution of Inactive Pyritic Tailings in the Elliot Lake Uranium District. Canadian Geotechnical Journal, Vol. 22, p. 110-128.

Dunn, R.W., Melis, L.A. and Vivyurka, A.J. 1977. The Pollution Control Program of Rio Algom Mines Limited in the Elliot Lake Region. Part I. Tailings Revegetation; Part 2. Seepage Treatment and Control. Rio Algom Mines Ltd.

Ecker, R.M. 1984. Effects of Rock Riprap Design Parameters on Flood Protection Costs for Uranium Tailings Impoundments, Pacific Northwest Laboratories, NUREG/CR-3751.

Edwards, W.M. 1967. Infiltration of Water into Soils as Influenced by Surface Conditions. Unpub. Ph.D. Thesis, Iowa State University, Ames, Iowa.

Eldorado Resources Limited, 1983. Volumes I to III of the Close-Out Studies for the Beaverlodge Mine (Volume II by Steffen Robertson and Kirsten Inc.).

Elmore, M.R. and Hartley, J.N., 1984. Laboratory Testing of Chemical Stabilizers for Control of Fugitive Dust Emissions from Uranium Mill Tailings. Report PNL 5025/NUREG/CR 3697, Pacific Northwest Laboratory, U.S. Nuclear Regulatory Commission, Washington, D.C.

Elmore, M.R. and Hartley, J.N. 1985. Evaluation of Field-Tested Fugitive Dust Control Techniques for Uranium Mill Tailings Piles Report PNL 5340/NUREG/CR-4089, Pacific Northwest Laboratory. U.S. Nuclear Regulatory Commission, Washington, D.C.

Evangelou, V.P. and Phillips, R.E., 1984. Ionic Composition of Pyritic Coal Spoil Leachate: Interactions and Effect on Saturated Hydraulic Conductivity. Reclamation and Revegetation Research, 3:65-76.

Evangelou, V.P., Phillips, R.E. and Shepard, J.S., 1982. Salt Generation in Pyritic Coal Spoils and Its Effect on Saturated Hydraulic Conductivity. Soil Science Society America J., 46:457-460.

Falk, J.A., Abt, S.R. and Nelson, J.D. 1985. Prediction of Gully Incision on Reclaimed Tailings Slopes, Submitted to the U.S. Nuclear Regulatory Commission, Report No.: CER85-86JAF-SRA-JDN6, pp. 76, September.

Federal Emergency Management Agency in Cooperation with Department of Civil Engineering, 1984. Sponsored the Stanford Workshop on Risk-Based Approach to Dam Safety Assessment, University of Colorado at Denver, Jan. 9-10, Denver, CO.

Foley, M.G., Kimball, C.S., Myers, D.A., Doesburg, J.M. 1985. The Selection and Testing for Armouring Uranium Tailings Impoundments, NUREG/CR-3747, US Nuclear Regulatory Commission.

Foster, G.R. and Meyer, L.D. 1972. A Closed-Form Erosion Equation for Upland Areas. In Sedimentation: Symposium to Honor Professor H.A. Einstein, ed. Shen, Ft. Collins, Colorado.

Foster, G.R., Lane, L.J., Nowlin, J.D., Laflen, J.M. and Young. R.A. 1980. A model to estimate sediment yield from field sized areas: Development of model. In: CREAMS-a field scale model for Chemicals, Runoff, and Erosion from Agricultural Management Systems. Volume I: Model Documentation, Chapter 3. USDA Science and Education Administration Conservation Report No. 26, pp. 36-64.

Frankel, E.G. 1984. Systems Reliability and Risk Analysis, Martinus Nijhoff Publishers, Kluwer Boston, Inc. 423 p.

Fredlund, D.G. 1985. Stability Analyses of Waste Management Structures, Seminar on Engineering of Waste Management Systems, Saskatoon, August.

Freeze, R.A., 1971. Three-Dimensional, Transient, Saturated-Unsaturated Flow in a Groundwater Basin. Water Resour. Res. 7:347-366.

Gardiner, R.T. 1977. Tailings Revegetation Experience at Cominco Ltd., Metal Mines in B.C. and the N.W.T. Proceedings 1st Annual Meeting British Columbia Mine Reclamation Symposium, Vernon, B.C., pages 35-54. B.C. Ministry of Mines and Petroleum Resources, Victoria, B.C.

Gill, W.R. 1959. The Effects of Drying on the Mechanical Strength of Lloyd Clay. Soil Sci. Soc. Amer. Proc. 23:225-257.

Gillen, D., Baldwin, J.S., Campbell, A.W., Hinkle, N.E., Pin, F.G., and Stauls, W.P. 1985. Summary of the Waste Management Programs at Uranium Recovery Facilities as they Relate to the 40CFS Part 192 Standards, NRC publication.

Golder Associates, 1984. Synthetic Liner Study from NUTP Report No. 841-3015.

Graf, W.H. 1971. Hydraulics of Sediment Transport. McGraw-Hill Book Co., New York, NY. 544 pp.

Grandt, A.F., 1978. Reclaiming Mined Land in Illinois for Row Crop Production. Journal of Soil and Water Conservation, 34:242-244.

Green, W.H. and Ampt, G.A. 1911. Studies on Soil Physics - I. The Flow of Air and Water Through Soils, J. Agric. Sci., Vol. 4., pp. 1-24.

Gregory, J.M., Johnson, H.P. and Kirkham, D. 1977. Soil Loss Equation: Derivation for Steep Slopes. ASAE Paper 77-2525, Presented at ASAE Winter Meeting, Chicago, Illinois.

Gregory, J.M., Johnson, H.P. and Kirkham, D. 1977. Infiltration Equation: Derivation to Include Surface Effects. ASAE Paper 77-2552. Presented at ASAE Winter Meeting, Chicago, Illinois.

Gregory, J.M. 1977. A Land-Phase Hydraulic Model for Strip Mined Lands. Unpub. Ph.D. Thesis, Iowa State University, Ames, Iowa.

Gregory, J.M., Steichen, J.M., and Mail, M.E. 1979. Infiltration Influenced by Tillage, Part 2 Infiltration Equation. ASAE Paper 79-2043. Presented at ASAE Summer Meeting, Winnipeg, Canada.

Gulf Minerals Canada Ltd., (1981). Environmental Impact Statement, Collins Bay B-Zone Development, Rabbit Lake Operations.

Haan, C.T. and Barfield, B.J. 1978. Hydrology and Sedimentology of Surface Mined Lands. Office of Continuing Education and Extension. University of Kentucky, Lexington, Kentucky.

Haan, C.T., Johnson, H.P. and Brarensier, O.L. 1982. Hydrologic Modelling of Small Watersheds. ASAE Monograph No. 5, St. Joseph, Michigan.

Hall, R.L. 1980. Land Reclamation with Trees in Iowa. In Anon, (ed), Trees for Reclamation. A Symposium sponsored by the Interstate Mining Compact Commission and U.S.D.A. Forest Service. Northeast Forest Experiment Station, Broomall, Pa. U.S.D.A. For. Serv. Gen. Tech. Rep. NE-61.

Hamaguchi, R. and McArthur, R. 1985. Final Abandonment Reclamation Plan - Revised June, 1985. MacLaren Forest Products Inc., Babine Mining Division, Granisle, B.C.

Hamon, W.I. 1980. Personal communication.

Hamon, W.R. and Bonta, J.B. 1979. Infiltration and Surface Mine Reclamation. ASAE Paper No. 79-2521. ASAE Winter Meeting, New Orelan, LA.

Hanks, R.J. and Ashcroft, G.L. 1980. Applied Soil Physics, . Advanced Series in Agricultural Sciences No. 8. Springer-Verlag, Berlin.

Hanks, R.J. and Bowers, S.A. 1962. A Numerical Solution of the Moisture Flow Equation for Infiltration into Layered Soils. Soil Sci. Am. Proc. Proc. 26:530-534.

Hanney, K.E., Jurgens, E.I., Comrie, D.C. and Fletcher, R.H. 1980. Reclamation Concepts and Practice for Uranium Tailings Impoundments. First International Conference on Uranium Waste Disposal, Vancouver. pp. 521-554.

Hantush, M.A. Hydraulics of Wells, Advances in Hydroscience, Vol. 1, Academic Press, New York, 1964.

Harr, M.E. 1962. Groundwater and Seepage, McGraw-Hill Book Co., 315 pp.

Harr, M.E. 1977. Mechanics of Particulate Media - A Probabilistic Approach. McGraw-Hill Book Co.

Harr, M.E. 1984. Reliability-Based Design in Civil Engineering, Shaw Lecture, Dept. of Civil Engineering, North Carolina State University, 68 p.

Havens, R. and Dean, K.C., 1969. Chemical Stabilization of the Uranium Tailings at Tuba City, Arizona. Bureau of Mines Report of Investigation 7288, U.S. Department of the Interior, Washington, D.C.

Haverkamp, R., Vauclin, M., Touma, J., Wierenga, P.J. and Vachaud, C. 1977. A Comparison of Numerical Simulation Models for One-Dimensional Infiltration. Soil Sci. Soc. Am. Proc. 41:285-294.

Hayes, J.C., Barfield, B.J. and Barnhisel, R.I. 1979. Filtration of Sediment by Simulated Vegetation II: Unsteady Flow with Nonhomogeneous Sediment. Trans. ASAE, Vol. 22(5):1063-1067.

Hillel, D. 1964. Infiltration and Runoff as Affected by Surface Crusts. Trans. VIIIth. International Soil Sci. Congr., Bucharest, Rumania, pp. 53-60.

Hillel, D. 1971. Soil and Water - Physical Principles and Processes. Academic Press, New York.

Hillel, D. 1980. Fundamentals of Soil Physics, Academic Press, New York.

Hillel, D. and Gardner, W.R. 1969. Steady Infiltration into Crust-Topped Profiles. Soil Sci. 108:2:137-142.

Hirschi, M.C. 1981. Efficiency of Small Scale Sediment Controls. Ag. Engr. File Report, University of Kentucky, Lexington, Kentucky.

Hoek, E. and Bray, J.W. 1977. Rock Slope Engineering, published by the Institute of Mining and Metallurgy, London.

Holtan, H.N. 1961. A Concept for Infiltration Estimates in Watershed Engineering. USDA-ARS Paper No. 41-51.

Holtan, H.N., England, C.B. and Shanholtz, V.O. 1967. Concepts in Hydrologic Soil Grouping. Trans. ASAE 10:3:407-410.

Holtan, H.N. and Creitz, N.R. 1967. Influence of Soils, Vegetation and Geomorphology on Elements of the Flood Hydrograph. Proc. Symposium on Floods and Their Computation, Leningrad, Russia.

Holtan, H.N. and Lopez, N.C. 1973. USDAHL-73 Revised Model of Watershed Hydrology, Plant Phys. Inst. Report No. 1.

Holtan, H.N. and Lopez, N.C. 1975. USDAHL-74 Revised Model of Watershed Hydrology. USDA-ARS TB-1518.

Horton, R.E. 1939. Analysis of Runoff Plant Experiments with Varying Infiltration-Capacity. Trans. Am. Geophysical Union, Hydrology Papers:693-711.

Hudson, N. 1971. Soil Conservation. Cornell University Press, Ithaca, N.Y.

Huff, F.A. 1967. Time Distribution of Rainfall in Heavy Storms. Water Resources Research 3(4):1007-1019.

Huggins. L.F. and Monke, E.J. 1967. A Mathematical Model for Simulating the Hydrologic Response of a Watershed. Water Resources Research 4:29-539.

Humphries, R.N. 1979. Landscaping Hard Rock Quarry Faces. Techniques No. 34. Landscape Design, August, p. 34-37.

Hwang, C.T., Murray, D.M. and Brooker, E.W., 1972. A Thermal Analysis of Structures on Permafrost. Canadian Geotechnical Journal, Vol. 9, No. 1.

IAEA, 1981. Current Practices and Options for Confinement of Uranium Mill Tailings. Technical Report No. 209.

Idike, F.I. 1977. Experimental Evaluation of Infiltration Equations. Unpubl. M. Sci. Thesis, University of Minnesota, St. Paul, Minnesota.

Idike, F.I., Larson, C.L., Slack, D.C. and Young, R.A. 1977. Experimental Evaluation of Two Infiltration Equations. Paper No. 77-2558. Presented at the ASAE Winter Meeting, Chicago, Illinois.

Ingles, O.G. and Metcalf, J.B. 1973. Soil Stabilization Principles and Practice. John Wiley and Sons, New York, N.Y.

Interstate Mining Compact Commission and USDA Forest Service. Northeast Forest Experiment Station, Broomall, Pa. USDA Forest Serv. Gen. Tech. Rep. NE-61.

Israelsen, W.W. and Hansen, V.E. 1967. Irrigation Principles and Practices, pp. 198-200. John Wiley: New York.

Israelsen, C.E., et al. 1980. Erosion Control During Highway Construction: Manual on Principles and Practices. National Cooperative Highway Research Program Report 221, Transportation Research Board, National Research Council, Washington, DC.

Johnson, K., 1985. Geochemistry of Uranium Wastes, #R-8502.

Kalin, M. 1977. Hydraulic Piping - Theoretical and Experimental Findings, Canadian Geotechnical Journal, Vol. 14, No. 1, February, pp. 107-124.

Kalin, M. 1981. A Preliminary Assessment of the Environmental Conditions of Two Abandoned Uranium Mill Tailings Sites in Saskatchewan. Submitted to Saskatchewan Environment and Environment Canada. Surveillance Report EPS-5-WNR-81-1.

Kalin, M. 1983. Long-Term Ecological Behaviour of Abandoned Uranium Mill Tailings. 1. Synoptic Identification of Invading Biota. Technology Development Report EPS 4-ES-83-1, Environmental Protection Service, Environment Canada, Ottawa.

Kalin, M. 1984. Long-Term Ecological Behaviour of Abandoned Uranium Mill Tailings - 2. Growth Patterns of Indigenous Vegetation on Terrestrial and Semi-Aquatic Areas. Technology Development Report EPS 3-HA-2. Environmental Protection Service, Environment Canada, Ottawa.

Kent, D.D. and Hwang, C.T. 1979. Use of a Geothermal Model in Northern Municipal Projects. Symposium on Utilities Delivery in Northern Regions. Edmonton.

Key Lake Mining Corp., 1982. Final Safety Report. Submitted to Atomic Energy Control Board of Canada in Support of an Application by Key Lake Mining Corporation for a Mining Facility Operating License.

Kezdi, A. 1976. Process of Hydraulic Soil Failure, Acta Technica Acad. Sci. Hungary, Vol. 82, No. 3-4, pp. 245-254.

Khan, H.R. 1971. Laboratory Study of River Morphology, Ph.D. Dissertation, Colorado State University, 189 pp.

Khosla, A.N., Bose, N.K. and Taylor, McKenzie E. 1936. Design of Weirs on Permeable Foundations, Central Board of Irrigation, Publication No. 12, India, 200 pp.

Kirkham, D. and Feng, C.L. 1949. Some Tests of the Diffusion Theory, and Laws of Capillary Flow, in Soils. Soil Sci. 67:29-40.

Kirpich, Z.P., 1940. Time of Concentration of Small Agricultural Watersheds, Civil Engineering, Vol. 10, No. 6, p. 632, June.

Kisch, M. 1959. The Theory of Seepage from Clay-Blanketed Reservoirs, Geotechnique, London, England, Vol. IX, pp. 22-28.

Klute, A. and Heerrann, D.F. 1978. Water Movement in Uranium Mill Tailings Profiles. TN ORP'LV-78-8. U.S. Environmental Protection Agency, Las Vegas, Nevada.

Knight and Piesold Ltd. 1981. Tailings Storage Facility. Report to Key Lake Mining Corporation on Key Lake Project - Report on Detailed Design - Site C, Volume 1.

Knight and Piesold Ltd. 1985. Monitoring of Layered Uranium Tailings - Phase I. DSS-NUTP Report #26SQ.23241-4-1679.

Knisel, W.G., Editor. 1980. CREAMS: A Field-Scale Model for Chemical, Runoff, and Erosion from Agricultural Management Systems. USDA, Conservation Research Report No. 26.

Konrad, J.M. and Morgenstern, N.R. 1980. A Mechanistic Theory of Ice Lens Formation in Fine-Grained Soils. Canadian Geotechnical Journal 17.

Konrad, J.M. and Morgenstern, N.R. 1981. The Segregation Potential of a Freezing Soil. Canadian Geotechnical Journal 18.

Konrad, J.M. and Morgenstern, N.R. 1982a. Prediction of Frost Heave in the Laboratory during Transient Freezing. Canadian Geotechnical Journal 19.

Konrad, J.M. and Morgenstern, N.R. 1982b. Effects of Applied Pressure on Freezing Soils. Canadian Geotechnical Journal 19.

Konrad, J.M. and Morgenstern, N.R. 1984. Frost Heave Prediction of Chilled Pipelines Buried in Unfrozen Soils. Canadian Geotechnical Journal, Vol. 21, No. 1.

Koussis, A.D. 1978. Theoretical Estimations of Flood Routing Parameters. Proc. of ASCE, Journal of Hydraulics, Vol. 104, (HY1):109-115.

Lambe, T.W., Marr, W.A. and Francisco, S. 1981. Safety of a Constructed Facility: Geotechnical Aspects, Journal of Geotechnical Eng. Eng. Div., ASCE, Vol. 107, No. GT3, March, pp. 339-352.

Lane, E.W. 1935. Security from Under-Seepage: Masonry Dam on Earth Foundations, Trans. ASCE, pp. 1235-1272, Discussions, pp. 1273-1351.

Lawson, D.W. 1986. Options for Reclaiming the Gunnar Uranium Tailings, Saskatchewan, Canada. Proc. 8th Annual Symp. Geotechnical and Geohydrological Aspects of Waste Management, Fort Collins, Co., pp. 339-350.

Leach, W. 1930. A Preliminary Account of the Vegetation of Some Non-Calcareous British Screes. Journal of Ecology, 18:321-334.

Leopold, L.B., Wolman, M.G. and Miller, J.P., 1964. Fluvial Processes in Geomorphology, Frention, San Francisco.

Leopold, L.B., and Wolman, M.G., 1957. River Channel Patterns: Braided, Meandering, and Straight, U.S. Geological Survey Professional Paper 282-B.

Li, C.T., Elmore, M.R., and Hartley, M.R. 1983. Review of Fugitive Dust Control for Uranium Mill Tailings - Pacific Northwest Laboratories, NUREG-CR-2856.

Lindsey, C.G., Long, L.W., Begej, C.W. 1982. Long-Term Survivability of Riprap for Armoring Uranium Mill Tailings and Covers: A Literature Review, U.S. Nuclear Regulatory Commission, Pacific Northwest Laboratory - NUREG/CR-2642, PNL-4225.

Lindsey, C.G., Mishima, J., King, S.E. and Walters, W.H., 1983. Survivability of Ancient Man-Made Earthen Mounds: Implications for Uranium Mill Tailings Impoundments, NUREG/CR-3061, U.S. Nuclear Regulatory Commission.

Longworth, L.D. 1977. Restoration of the Mining Area at Captain's Flat. Australian Mining Industry Council Environmental Workshop Rehabilitation Papers (quoted by Hore-Lacey, I. 1978. Sulphide Ores: An Overview of the Effect of Mining. In Rummery, R.A. and Howes, N.M.W. (Editors) Management of Land Affected by Mining, CSIRO, Division of Land Resources Management, Canberra, Australia.

Lumb, P. 1966. Variability of Natural Soils, Can. Geot., Journal, Vol. 3, No. 2, May.

Lutz, J.F. 1952. Mechanical Impedance and Plant Growth, p. 43-71. In B.T. Shaw (Ed.) Soil Physical Conditions and Plant Growth. Agron. Monograph, Vol. II, Academic Press, New York.

McArthur, R. 1985. Annual Reclamation Report for 1984: Reclamation Permit M-35. MacLaren Forest Products Inc., Babine Mining Division, Granisle, B.C. February.

McElroy, A.D., et al. 1976. Loading Functions for Assessment of Water Pollution from Nonpoint Sources. Midwest Research Institute, Kansas City, Missouri.

McInnis, P. 1985. Literature Review for the National Uranium Tailings Program (NUTP) - Long -Term Effects of Natural Events on Uranium Tailings Impoundments. Prepared for Department of Energy, Mines and Resources, Canadian Centre for Mineral and Energy Technology. National Uranium Tailings Program. NUTP 85-3(TR).

MacLaren Associates, and Golder Associates, 1981. Hydrogeological Assessment of the Tailings Impoundment Basin. Report to Rio Algom Ltd. on Quirke Mine Tailings Impoundment.

McWhorter, D.B. 1985. Seepage in the Unsaturated Zone: A Review, in Seepage and Leakage from Dams and Impoundments, R.L. Volpe and W.E. Kelly, Ed., Proceedings of Geotechnical Engineering Division Symposium, ASCE, Denver May 5, pp. 200-219.

McWhorter, D.B., and Nelson, J.D. 1979. Unsaturated Flow Beneath Tailings Impoundments. Journal of the Geotechnical Division, ASCE, Vol. 105, No. GT11, Nov., pp. 1317-1334.

McWhorter, D.B., Rowe, J.W., Rowe, Van Liew, M.W., Chandler, R.L., Skogerboe, R.K., Sunada, D.K., and Skogerboe, G.V. 1979. Surface and Subsurface Water Quality Hydrology in Surface Mined Watersheds. Part I. Text. EPA report EPA-600/7-79-193a, Cincinnati, Ohio.

Mannering, J.V. 1967. The Relationship of Some Physical and Chemical Properties of Soils to Surface Sealing. Unpub. Ph.D. Thesis, Purdue University, Lafayette, Indiana.

Marsal, R.J. and Pohlenz, W, 1972. The Failure of Laguna Dam, Proc. Specialty Conf. on Performance of Earth and Earth-Supported Structures, ASCE, Purdue University, June 11-14, Vol. 1, Part 1, pp. 489-505.

Marshall, I.B.. 1982. Mining, Land Use and the Environment, 1:A. Canadian Overview. Lands Directorate, Environment Canada.

Marshall, I.B. 1982. Mining, Land Use and the Environment. II. A Review of Mine Reclamation Activities in Canada. Land Use in Canada Series, No. 23, Lands Directorate, Environment Canada, Ottawa, pp. 288.

Marsland, A. 1953. Model Experiments to Study the Influence of Seepage on the Stability of a Sheeted Excavation in Sand, Geotechnique, Vol. 3, No. 2, pp. 233-240.

Matthews, M.L. 1981. The Research Effort of the Uranium Mill Tailings Remedial Action Program, Symp. on Uranium Mill Tailings Management, Fort Collins, Co., October 26-27.

Mayer, D.W. and Zimmerman, D.W. 1981. Radon Diffusion Through Uranium Mill Tailings and Cover Defects - Pacific Northwest Laboratory, NUREG/CR-2457.

Meeuwig, R.O. 1971. Infiltration and Water Repellancy in Granitic Soils. U.S. Forest Serv. Res. Paper Int.-111.

Mein, R.G. and Larson, C.L. 1971. Modeling Infiltration Component of the Rainfall-Runoff Process. Bulletin 43, Water Resour. Research Center, University of Minnesota, Minneapolis, Minnesota, 72 pp.

Mein, R.G. and Larson, C.L. 1973. Modeling Infiltration During a Steady Rain. Water Resour. Res. 9(2):284-394.

Meyer, L.D. and Wischmeier, W.H. 1969. Mathematical Simulation of the Process of Soil Erosion by Water. TRANSACTIONS of the ASAE 12(6):754-758, 762.

Meyer, L.D., Harmon, W.C. and McDowell, L.L. 1978. Sediment Sizes Eroded from Rowcrop Sideslopes. ASAE Paper No. 78-2518, ASAE, St. Joseph, MI 49085, 20 pp.

Michelutti, R.E. 1978. The Establishment of Vegetation on High Iron-Sulphur Mine Tailings by Use of an Overburden. Proceedings of the 3rd Annual Meeting of the Canadian Land Reclamation Association, Laurentian University, Sudbury, Ontario, pp. 25-30. Canadian Land Reclamation Association, Guelph, Ontario.

Miller, D.E. and Gardner, W.H. 1962. Water Infiltration into Stratified Soil. Soil Sci. Soc. Am. Proc. 26:115-118.

Mine Waste Reclamation Ltd., 1980a. Survey of Plant Growth on Nordic, Pronto and Lacnor Tailings Disposal Areas in May 1980, with Addendum. Report No. 1 to Rio Algom Mines Ltd.

Mine Waste Reclamation Ltd., 1980b. Results and Interpretation of Soil Test Analysis on Tailings Samples from Nordic, Lacnor and Pronto Tailings Disposal Areas. Report No. 2 to Rio Algom Mines Ltd.

Mine Waste Reclamation Ltd., 1980c. Review of Literature on the Applicability of Plant Tissue Analysis for Determining Nutrient Status of Tailings Vegetation and Design of Annual Fertilizer Programs. Report No. 3 to Rio Algom Ltd.

Mine Waste Reclamation Ltd., 1980d. Supplementary Reports on Results and Interpretation of Soil Test Analysis and Vegetation Composition on Tailings from Nordic, Pronto and Lacnor Tailings Disposal Areas. Report No.4 to Rio Algom Ltd.

Mine Waste Reclamation Ltd., 1981. Evaluation of Plant Growth Potential on Uranium and Copper Tailings, Rio Algom Ltd., Elliot Lake, Ontario. Unpublished report prepared for Rio Algom Ltd. by Mine Waste Reclamation.

Mine Waste Reclamation Ltd. 1982a. Evaluation of Plant Growth Potential of Overburden Materials AMOK Ltd. - Cluff Mine, Saskatchewan. Unpublished Report Prepared for AMOK Ltd.

Mine Waste Reclamation Ltd., 1982b. Growth Room Evaluation of Soils and Granular Materials as Media for Plant Growth - Upper Salmon Hydro-Electric Development, Newfoundland. Unpublished report prepared for Acres Consulting Services, Niagara Falls.

Mine Waste Reclamation, 1982c. Potential for Vegetative Growth on Waste Smelter Slag at Brunswick Smelting, Belledune, New Brunswick. Report to Brunswick Mining and Smelting - Smelting Division.

Mine Waste Reclamation Ltd. 1985. Evaluation of Plant Growth Potential on Dredged Sandstone Rock Berms at Sault Ste. Marie. Unpublished Report Prepared for Parks Canada - Ontario Region, Cornwall, Ontario.

Minister of Supply and Services. 1978. Hydrological Atlas of Canada, Printing and Publishing Supply and Services, Ottawa.

Mitchell, A.R. 1969. Computational Methods in Partial Differential Equations. John Wiley and Sons, New York.

Molz, F.J. and Remson, I. 1970. Application of an Extraction-Term Model to the Study of Moisture Flow to Plant Roots.

Moore, I.D. and Larson, C.L. 1979. Effects of Drainage Projects on Surface Runoff from Small Depressional Watersheds in the North Central Region. WRRC, University of Minnesota Gra. School Bull, 99.

Moore, I.D. 1979a. Infiltration into Tillage Affected Soils. Ph.D. Thesis, University of Minnesota, St. Paul, Minnesota.

Moore, I.D. 1979b. Green-Ampt and Mein-Larson Two-Stage Infiltration Equations Modified for Surface Effects. Agricultural Engineering Technical Series Report no. 16, University of Kentucky, Lexington, Kentucky.

Moore, I.D. 1980a. Effect of Surface Sealing on Filtration. Paper No. 80-2524. Presented at the ASAE Winter Meeting at Chicago, Illinois.

Moore, I.D. 1980b. Modelling the Effects of Surface Sealing on Infiltration. Unpub. Research Project, Agricultural Engineering Department, University of Kentucky, Lexington, KY.

Moore, I.D. 1981. Infiltration Equations Modified for Surface Effects. Journal of the Irrigation and Drainage Division, ASCE, Vol. 107, No. IRI:71-86.

Moore, I.D. and Eigel, J.D. 1981. Infiltration into Two-Layered Soil Profiles. Transactions of the American Society of Agricultural Engineers (accepted for publication).

Morel, F., and Morgan, J.J. 1972. A Numerical Method for Computing Equilibria in Aqueous Chemical Systems. Environ. Sci. Tech. 6:58-67.

Morel-Seytoux, H.J. 1973. Two Phase Flows in Porous Media. Advances in Hydroscience, 9:110-202.

Morel-Seytoux, H.J. and Khanji, J. 1974. Derivation of an Equation of Infiltration. Water Resour. Res. 10(4):795-800.

Morgenstern, N.R. and Nixon, J.F. 1971. One Dimensional Consolidation of Thawing Soils. Canadian Geotechnical Journal, Vol. 20, No. 4.

Morin, K.A., Cherry, J.A., Lim, T.P., and Vivyurka, A.J. 1982. Contaminant Migration in a Sand Aquifer Near an Inactive Uranium Tailings Impoundment, Elliot Lake, Ontario. Canadian Geotechnical Journal, Vol. 19, No. 1.

Morin, K.A., 1985. 1984 Geochemical Study of the Acidic Contaminant Plume Near the Nordic Main Uranium - Tailings Impoundment, Elliot Lake, Ontario. DSS File No.: 26SQ.23241-4-1689. Serial No.: OSQ84-00229. The National Uranium Tailings Program.

Mosley, M.P. 1971. An Experimental Study of Rill Erosion, M.S. Thesis, Colorado State University, Fort Collins, CO, pp. 99.

Murray, D.R. 1977. Reclamation by Vegetation. Pit Slope Manual, Supplement 10-1, Volume 1 - Mine Waste Description and Case Histories, CANMET, Department of Energy, Mines and Resources, Ottawa.

Murray, D.R. 1977. The Influence of Uranium Mine Tailings on Tree Growth at Elliot Lake, Ontario. Proceedings 2nd Annual General Meeting Canadian Land Reclamation Association, Edmonton, Alberta. Canadian Land Reclamation Association, Guelph, Ontario.

Murray, D.R. and Moffett, D. 1977. Vegetating the Elliot Lake Uranium Mine Tailings. Journal of Soil and Water Conservation, July-August, 29-30.

Murray, D., Webber, B., and LaRocque, E. 1977. Reclamation of Lower Williams Lake Tailings Area of Denison Mines Limited. Energy, Mines and Resources Canada, Minerals Research Program.

Nakjima, Y. 1968. Study of the Failure of Piping in Polder Dikes for Land Reclamation, Soils and Foundations, Vol. VIII, No. 4, pp. 30-47.

Nelson, J.D., Volpe, R.C., Wardwell, R.E., Schumm, S.A. and Staub, W.P. 1983. Design Considerations for Long-Term Stabilization of Uranium Mill Tailings Impoundments, prepared for U.S. Nuclear Regulatory Commission, NUREG/CR 3397, pp. 20, October.

Nelson, J.D. 1978. Evaluation of Long-Term Stability of Uranium Mill Tailings Disposal Alternatives. Argonne National Laboratories - April.

Nielsen, D.R., Biggar, J.W., and Erh, K.T. 1973. Spatial Variability of Field-Measured Soil-Water Properties. Hilgardia:43:215-259.

Nielsen, D.R., Kirkham, R.D. and VanWiyk, W.R. 1961. Diffusion Equation Calculations of Field Soil Water Infiltration Profiles. Soil Sci. Amer. Proc. 25:165-168.

Nielson, R.F. and Peterson, H.B. 1978. Vegetating Mine Tailings Ponds. In: "Reclamation Drastically Disturbed Lands" Edited by F.W. Schaller and P. Sutton, pages 645-652.

Nixon, J.F. and McRoberts, E.C., 1973. A Study of Some Factors Affecting the Thawing of Frozen Soils. Canadian Geotechnical Journal, Vol. 10, No. 3.

Norsk Hydro, 1972 (Hillestad, K.D., 1973) Sprengstein Tipp og Landskap Kraft Cg Miljo 2. Oslo: Norges Vassdrgas-og elektrisitetsvesen.

O'Brien, P.D. 1981. The Umtrap Technology Development Program: A Progress Report", Symp. on Uranium Mill Tailings Management, Fort Collins, Co., October 26-27.

Overton, D.E. 1971. Estimation of Surface Water Lag Time from the Kinematic Wave Equation. Water Resources Bulletin, 7(3):428-440.

Overton, D.E. and Meadows, M.E. 1976. Storm Water Modeling. Academic Press, New York.

Overton, D.E. and Troxler, W.L. 1978. Regionalization of Stormwater Response. Paper presented at the American Geophysical Union Meeting, Miami, Florida, April 17-21.

Overton, D.E. and Crosby, E.C. 1979. Effects of Contour Coal Strip Mining on Stormwater Runoff and Quality. Report to U.S. Department of Energy, Department of Civil Engineering, University of Tennessee, Knoxville, Tennessee.

Pall, R., Jarrett, A.R. and Morrow, C.T. 1979. Two-Dimensional Infiltration by an Explicit Method for Presentation at the 1979 Winter Meeting ASAE December 11-14.

Parkhurst, D.L., Thorstenson, D.C. and Plummer, L.N. 1980. PHREEQE - A Computer Program for Geochemical Calculations. U.S. Geol. Survey Water Resources Invest. 80-96, 210 p.

Parlange, J.Y. 1971. Theory of Water Movement in Soils: I. One-Dimensional Absorption. Soil Sci. Vol. 111, No. 2.

Parlange, J.Y. 1971. Theory of Water Movement in Soils: 2, One-Dimensional Infiltration. Soil Sci. 111(3):170-174.

Parlange, J.Y. 1975a. Theory of Water Movement in Soils: II Conclusion and Discussion of Some Recent Developments. Soil Sci. Vol. 119, No. 2, 1975.

Parlange, J.Y. 1975b. A Note on the Green-Ampt Equation. Soil Sci. 119(6):460-467.

Parlange, J.Y. and Babu, D.K. 1976a. A Comparison of Techniques for Solving the Diffusion Equation with an Exponential Diffusivity. Water Resources Research Vol. 12, No. 6.

Parlange, J.Y. and Babu, D.K. 1976b. On Solving the Infiltration Equation: A Comparison of Perturbation and Interactive Techniques. Water Resources Research Vol. 12, No. 6.

Patton, P.C. and Schumm, S.A. 1975. Gully Erosion, Northwestern Colorado: A Threshold Phenomenon. Geology, Vol. 3, pp. 88-90.

Pedersen, T.A., Rogowski, A.S. and Pennock, R. 1980. Physical Characteristics of Some Minespoils. Soil Sci. Soc. Am. J., 44(2):321-328.

Perrens, A.J. and Watson, K.K. 1977. Numerical Analysis of Two-Dimensional Infiltration and Redistribution. Water Resources Research, 13(4):781-790.

Peters, T.H. 1978. INCO Metals Reclamation Program. Proceeding 3rd Annual Meeting Canadian Land Reclamation Association, pages 31-38, Laurentian University, Sudbury. Canadian Land Reclamation Association, Guelph.

Philip, J.R. 1957a. The Theory of Infiltration. 1. The Infiltration Equation and its Solution. Soil Sci. 83:345-357.

Philip, J.R. 1957b. The Theory of Infiltration. 2. The Profile at Infinity. Soil Sci. 83:435-448.

Philip, J.R. 1957c. The Theory of Infiltration. 3. Moisture Profiles and Relation to Experiment. Soil Sci. 84:163-178.

Philip, J.R. 1957d. The Theory of Infiltration. 4. Sorptivity and Algebraic Infiltration Equations. Soil Sci. 84:257-264.

Philip, J.R. 1957e. The Theory of Infiltration. 5. The Influence of Water Content. Soil Sci. 84:329-339.

Philip, J.R. 1957f. Numerical Solution of Equations of the Diffusion Type with Diffusivity Concentration Dependent. II. Australian J. Phy. 10:29-42.

Pit Slope Manual, 1977. Chapter 9 - Waste Embankments. Publication by CANMET, Energy, Mines and Resources Canada.

Pritchett, W. 1979. Properties and Management of Forest Soils. J. Wiley & Sons, New York.

Rafferty, P.J., 1981. NEA Program on the long-Term Aspects of Uranium Mill Tailings Management and Disposal. Symp. on Uranium Mill Tailings Management, Fort Collins, Co., October 26-27.

Reuss, F.F., and Schattenberg, J.W. 1972 Internal Piping and Shear Deformation, Victor Braunig Dam - San Antonio, Texas, Proc. of Specialty Conf. on Performance of Earth and Earth-Supported Structures, ASCE, Purdue University, June 11-14, Vol. 1, Part 1, pp. 627-651.

Richards, L.A. 1954. Diagnosis and Improvement of Saline and Alkali Soil. Agricultural Handbook No. 60. United States Department of Agriculture, Washington, D.C.

Richardson, J.A. 1977. High Performance Plant Species in Reclamation. In Hackett, B. (ed.). Landscape Reclamation Practice, IPC Science and Technology Press, Guildford, U.K.

Rogers, B.C., Nielson, K.K., Halkwarf, D.R. 1984. Radon Attenuation Handbook for Uranium Mill Tailings Cover Design - U.S. Nuclear Regulatory Commission, Pacific Northwest Laboratory, NUREG/CR-3533, PNL -4878.

Rose, H.E. 1945. An Investigation into the Laws of Flow of Fluids Through Beds of Granular Material. Proc. Inst. Mech. Eng. 153, 141-148.

Ross, B.B., Contractor, D.N., Li, E.A., Shanholtz, V.O. and Carr, J.C. 1976. A Model for Predicting Flood Hazards Due to Specific Land-Use Practices. Virginia Water Resources Research Center Bull. 99. Virginia Poly. Inst. and State Univ., Blackburg, Virginia.

Rowell, M.J. 1981. Assessment of the Soil Resource in the Reclamation of Disturbed Mountainous Areas. Proc. 6th Ann. Meeting Canadian Land Reclamation Association, Cranbrook, B.C., pp. 241-270. C.L.R.A., Guelph, Ontario.

Rubin, J., Steinhardt, R., and Reininger, P. 1964. Soil Water Relations During Rain Infiltration: II. Moisture Content Profiles During Rains of Low Intensities. Soil Sci. Soc. Am. Proc. 28:1-5.

Rummery, R.A. and Howes, K.M.W. 1978. (Editors). Management of Land Affected by Mining. CSIRO, Division of Land Resources Management, Canberra, Australia.

Russell, E.J. 1964. Soil Conditions and Plant Growth, Longmans Publishers, London, U.K.

Sandoval, F.M. and Gould, W.L., 1978. Improvement of Saline and Sodium Affected Disturbed Lands. In: Schaller, F.W. and Sutton, P. (Editors), Reclamation of Drastically Disturbed Lands. ASA/CSA/SSSA. Madison, Wisconsin, U.S.A.

Schulz, J.D., 1969. The Vertical Rooting Habit of Black Spruce, White Spruce and Balsam Fir. Ph.D. Dissertation, University of Michigan. University MicroFilms, Inc., Ann Arbour (quoted in Armson, K.A. 1977 Forest Soils: Properties and Processes), University of Toronto, Ontario).

Schuman, G.E. and Power, J.F. 1981. Topsoil Management of Mined Lands. Journal Soil and Water Conservation, 36: 77-78.

Schumm, S.A. 1977. The Fluvial System, Wiley, New York.

Schumm, S.A. and Chorley, R.J. 1983. Geomorphic Controls on the Management of Nuclear Waste, NRC Monograph, U.S. Nuclear Regulatory Commission, Washington, D.C.

Schumm, S.A., and Khan, H.R. 1972. Experimental Study of Channel Patterns, Geol. Soc. Am. Bull., Vol. 83, pp. 1755-1770.

Schultze, E. 1972. Frequency Distributions and Correlations of Soil Properties, Statistics and Probability in Civil Engineering, Hong Kong University Press, distributed by Oxford University Press, London.

Seelye, E.E., 1960. Design, 3rd Ed., Vol. 1, John Wiley and Sons, Inc., New York, NY.

Sentko, M. 1961. Der Zeitliche Ablauf des Schwimmsand aufbruches und der Einfluss der geometrischen Anordumg der Baugrubenumschliess ungen auf das kritische Gefalle, Ph.D. Thesis, Veroffentlichungen Institut fur Bodenmechanik, Technische Hochschule, Karlsruhe, No. 7, 130 pp.

Sharma, S.K. and Rao, J.V. 1969. Generalized Theory of Exit Gradients for the Stability of Sloping Surfaces in Porous Media Against Efflux of Seepage, Irrigation and Power, Journal of Central Board of Irrigation and Power, Vol. 26, No. 3, July, pp. 257-265.

Shaw, B.T., Haise, H.R. and Farnsworth, R.B. 1942. Four Years Experience with the Soil Penetrometer. Soil Sci. Soc. Amer. Proc. 7:48-55.

Shepherd, Thomas A. and Brown, Stephen E. 1982. A General Model of Contaminant Migration from Uranium Tailings Impoundments in Proceedings of the Fifth Symposium on Uranium Mill Tailings Management, Colorado State University, Fort Collins, Colorado, December 9-10, p. 241-258.

Simons, D.B. and Senturk, F. 1977. Sediment Transport Technology, Water Resources Publications, Fort Collins, Co. p. 456.

Singh, V.P. and Buapeng, S. 1977. Effect of Rainfall - Excess Determination on Runoff Computation: Water Resources Bull. Vol. 13, No.3.

Skaggs, R.W., Huggins, L.R., Monke, E.J. and Foster, G.R. 1969. Experimental Evaluation of Infiltration Equations. Trans. ASAE 12(6):822-828.

Skaggs, R.W. and Khaleel, R., 1979. Chapter 4 - Infiltration. In ASAE Monograph No. 4, Hydrologic Modeling of Small Watersheds. C. T. Haan et al., Ed. (In Press).

Smith, A.C.S. and Van Zyl, D. 1983. Design Criteria in Acid Generating Mine Waste Disposal. Proceedings of Seventh Panamerican Conference on Soil Mechanics and Foundation Engineering, Vancouver, B.C.

Smith, R.E. and Woolhiser, D.A., 1971. Mathematical Simulation of Infiltrating Watersheds. Hydrol. Paper 47, Colorado State Univ. Fort Collins. Co.

Smith, R.E. 1972. The Infiltration Envelope: Results from a Theoretical Infiltrometer, Journal of Hydrology, 17:1-21.

Smith, R.E. 1976. Approximations for Vertical Infiltration Rate Patterns. Transactions of the ASAE, 19(3):505-509.

Smith, R.E. and Parlange, J.Y. 1978. A Parameter-Efficient Hydrologic Infiltration Model. Water Resour. Res. 14(3):533-538.

Snodgrass, W.J., Lush, D.L. and Capobianco, J. Implications of Alternate Geochemical Controls of the Temporal Behaviour of Elliot Lake Tailings.

Soil Conservation Service. 1972. Hydrology Section 4, Soil Conservation Service National Engineering Handbook, U.S. Department of Agriculture, Washington, D.C.

Soil Conservation Service. 1973. A Method for Estimating Volume and Rate of Runoff in Small Watersheds. SCS-TP-149. U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C.

Soil Conservation Service. 1975a. Urban Hydrology for Small Watersheds. Technical Release No. 55, Soil Conservation Service, U.S. Department of Agriculture, Washington, D.C.

Soil Conservation Service. 1975b. Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas. Soil Conservation Service, College Park, Maryland.

Soil Conservation Service. 1975c. Hydrology. Section 4, Soil Conservation Service National Engineering Handbook, U.S. Department of Agriculture, Washington, D.C.

Solymar, Z.V. and Nunn, J.O.H., 1983. Frost Sensitivity of Core Materials: Case Histories. Canadian Geotechnical Journal. Vol. 20, No. 3., pp. 373-384.

Steffen Robertson & Kirsten, 1985. Evaluation of Flood Estimation Techniques. Phase 2 Report: An Evaluation of Hydrological Techniques for Making Flood Estimations on Small Ungauged Catchments. Water Research Commission of South Africa, Pretoria.

Stephenson, D., 1979. Rockfill in Hydraulic Engineering, Developments in Geotechnical Engineering, 27, Elsevier Scientific Publishing Company, pp. 50-60.

Stevenson, D.R. 1978. Hydrogeological Evaluation of Mill and Tailings Pond Sites, Key Lake Project. Report to Key Lake Mining Corp.

Sultan, H.A. and Fleming, P. 1974. Soil Erosion and Dust Control on Arizona Highways Part 1: State of the Art Review. Report ADUT-RS-10-141-I. Arizona Department of Transportation, Phoenix, Arizona.

Swanson, S. and Abouguendia, A. 1981. The Problem of Abandoned Uranium Tailings in Northern Saskatchewan: An Overview. SRC Publication No. C-805-48-C-81, Saskatchewan Research Council.

Swartzendruber, D. 1969. The Flow of Water in Unsaturated Soils. p. 215-292. In, R.M. DeWiest (Ed.) Flow Through Porous Media. Academic Press, New York.

Swartzendruber, D. and Youngs, E.G. 1974. A Comparison of Physically Based Infiltration Equations. Soil Sci. 117(3):165-167.

Taylor, H.M. and Gardner, H.R. 1963. Penetration of Cotton Seedling Taproots as Influenced by Bulk Density, Moisture Content and Strength of Soil. Soil Sci. 96:153-156.

Terzaghi, K. 1922. Der Grundbruch an Stauwerken und seine Verhutung Die Wasserkraft, Vol. 17, pp. 445-449, Reprinted in "From Theory to Practice in Soil Mechanics", Wiley, 1960, pp. 114-118.

Terzaghi, K. 1929. Effect of Minor Geologic Details on the Safety of Dams, American Institute of Mining and Metallurgical Engineers, Technical Publication 215, pp. 31-44, Reprinted in From Theory to Practice in Soil Mechanics. Wiley, 1960, p. 119-132.

Terzaghi, K. 1943. Theoretical Soil Mechanics, John Wiley & Sons, Inc., New York, 510 pp.

Thornthwaite, C.W. and Mather, J.R., 1957. Instructions and Tables for Computing Potential Evapotranspiration and the Water Balance. Publications in Climatology X: No. 3. Drexel Institute of Technology, Centerton, New Jersey.

Truesdell, A.H., and Jones, B.F. 1974. WATEQ, A Computer Program for Calculating Chemical Equilibria of Natural Waters. U.S. Geol. Survey J. Res. 2:233-248.

Turnbull, W.J. and Mansur, C.I. 1961. Investigation of Underseepage - Mississippi River Levees, Trans, ASCE, Vol. 126, Part I, pp. 1429-1482.

United States Department of Agriculture - Soil Conservation Service 1973. Kentucky Guide for Classification, Use and Vegetative Treatment of Surface Mine Soil. Lexington, Kentucky.

United States Department of Agriculture, 1947. Soil Conditions that Influence Wind Erosion, Technical Bulletin 1185.

United States Department of Agriculture, 1968. Agricultural Handbook #346, Wind Erosion Forces in the U.S. and Their Use in Predicting Soil Loss, U.S. Government Printing Office, Washington, D.C.

University of Kentucky. 1983. Design Manual for the Sedimot II Hydrology and Sedimentology Model, University of Kentucky, Lexington, Kentucky.

University of Saskatchewan, 1985. Seminar on Engineering of Waste Management Systems, Saskatoon, August.

U.S. Army Corps of Engineers. 1970. Hydraulic Design of Flood Control Channels, EM 1110-2-1601, July.

U.S. Army Corps of Engineers, 1971. Engineering and Design , Additional Guidance for Riprap Channel Protection, ETL-1110-2-120, Office of the Chief of Engineers, pp. 52.

U.S. Bureau of Reclamation. 1978. Hydraulic Design of Stilling Basins and Energy Dissipators, U.S. Department of the Interior, Engineering Monograph No. 25.

van Duin, R.H. 1955. Tillage in Relation to Rainfall Intensity and Infiltration Capacity of Soils. Netherlands J. Agr. Sci., Vol. 3, pp. 182-191.

Van Phuc, Le and Morel-Seytoux, H.J. 1972. Effect of Soil Air Movement and Compressibility on Infiltration Rates. Soil Sci. Amer. Proc. 36:237-241.

van Zyl, D. 1979. Seepage Erosion of Geotechnical Structures Subjected to Confined Flow - A Probabilistic Design Approach, Ph.D. Thesis, Purdue University, August, 244 pp.

van Zyl, D. and Harr, M.E. 1981. Seepage Erosion Analyses of Structures, Proc. of the 10th ICSMFE, Stockholm, Sweden.

Veerabhadra, Rao D. 1972. Tail Erosion and Piping Problem, Indian Geotechnical Journal, Vol. 2, No. 4, pp. 348-351.

Veihmeyer, F.J. and Hendrickson, A.H. 1948. Soil Density and Root Penetration. Soil Sci. 65:487-493.

Verry, E.S. 1976. Estimating Water Yield Differences Between Hardwood and Pine Forests. Research Paper NC-128, United States Forest Service, North Central Forest Experiment Station, St. Paul, Minnesota.

Verry, E.S. and Timmons, D.R. 1982. Waterborne Nutrient Flow Through an Upland-Peatland Watershed in Minnesota. Ecology 63(5):1456-1467.

Vick, S.G. 1983. Planning, Design and Analysis of Tailings Dams, John Wiley & Sons.

Voorhees, L.D., Sale, M.J., Webb, J.W. and Mulholland, P.J., 1983. Guidance for Disposal of Uranium Mill Tailings: Long-Term Stabilization of Earthern Cover Materials. Report ORNL/TM-8685/NUREG/CR-3199, Oak Ridge National Laboratory, U.S. Nuclear Regulatory Commission, Washington, D.C.

Walters, W.H. 1982. Rock Rip Rap Design Methods and Their Applicability to Long-Term Protection of Uranium Mill Tailings Impoundments. Report PNL-4252/NUREG/CR-2684, Pacific Northwest Laboratory, U.S. Nuclear Regulatory Commission, Washington, D.C.

Walters, W.H. 1983. Overland Erosion of Uranium Mill Tailings Impoundment: Physical Processes and Computational Methods. U.S. Nuclear Regulatory Commission, Pacific Northwest Laboratory - NUREG/CR-3027, PNL-4523.

Walters, W.H. and Skaggs, R.L. 1986. The Protection of Uranium Tailings Impoundments Against Overland Erosion, NUREG/CR-4323, U.S. Nuclear Regulatory Commission.

Ward, A.D., Bridges, T. and Wilson, B. 1981. A Simple Procedure for Developing A Design Storm Hydrograph. Paper No. 80099 of the Water Resources Bulletin, American Water Resources Association, Vol. 17, No. 2, Pp. 209-214, April.

Ward, A.D., Haan, C.T., and Tapp, J.S., 1979. The Deposits Sedimentation Pond Design Manual, Univ. of Kentucky, Lexington, Kentucky.

Ward, A.D., Wilson, B., Bridges, T. and Barfield, B. 1980. An Evaluation of Hydrologic Modelling Techniques for Determining a Design Storm Hydrograph. Proceedings of the 7th International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky.

Ward, A.D., Wells, L.G. and Phillips, R.E. 1983. Characterizing Unsaturated Hydraulic Conductivity of Western Kentucky Surface Mine Spoils and Soils. Soil Science Society of America Journal, Vol. 47, No.5: 847-854, Madison, WI.

Ward, A.D., Wells, L.G. and Phillips, R.E. 1983. Infiltration through Reconstructed Surface Mines Spoils and Soils. Transactions of the ASAE, Vol. 26, No. 3: 821-832, St. Joseph, Michigan.

Ward, A.D., Smith, A.C.S. and Caldwell, J.A. 1984. Surface Erosion and Sediment Control at Open-Cast Mines in Southern Africa. Proceedings of the Harare Symposium on Challenges in African Hydrology and Water Resources IAHS Publ. No. 144: 517-527.

Watkin, E.M. 1985. Revegetation of Hard Rock Waste from Mining and Construction Operations. Proceedings 10th Annual Meeting Canadian Land Reclamation Association, 18-21 August, Quebec City, pages 167-181. Canadian Land Reclamation Association, Guelph, Ontario.

Watkin, E.M. and Watkin, J. 1982. Tailings Reclamation in Eastern Canada. World Mining, December.

Watkin, E.M. and Watkin, J. 1984. Soil test Data Requires Careful Analysis. Canadian Mining Journal, March, 46-48.

Watkin, E.M. and Winch, J.E. 1973. Composted Organic Wastes. Sewage Sludges and Rock Phosphate for the Amelioration of Acid Uranium Tailings. Proceedings International Conference on Land for Waste Management, Ottawa, October 1973, pages 48-56. National Research Council of Canada, Ottawa.

Watson, J.P. and Luxmoore, R.J. 1979. Relevance of the European Open-Cast Reclamation Approach to American Surface Mining. Abstracts, American Council for Reclamation Research Meeting, University, Alabama - published in "Reclamation Review" 3, 61-62, 1980.

Watson, K.K. 1965. Some Operating Characteristics of a Rapid Response Tensiometer System, Water Resources Res. 1, 577-586 (1965b) Non-Continuous Porous Media Flow. Report 84, Water Resources Lab., University New South Wales, Manly Vale, N.S.W., Australia.

Whisler, F.D., Watson, K.K. and Perrens, S.J. 1972. The Numerical Analysis of Infiltration into Heterogeneous Porous Media. Soil Sci. Soc. Amer. Proc. 36:6:868-874.

Whitman, R.V. 1984. Evaluating Calculated Risk in Geotechnical Engineering, Journal of Geot. Eng., ASCE, Vol. 110, No. 2, Feb., pp. 145-188.

Williams, J.R. 1975. Sediment Routing for Agricultural Watersheds. Water Resources Bulletin 11(5):965-974.

Williams, J.R. 1978. A Sediment Yield Model Routing Model. In: Proc. of the Specialty Conference on Verification of Mathematical and Physical Models in Hydraulic Engineering. American Society of Civil Engineers, New York, NY, pp. 602-670.

Wilson, B.N., Slack, D.C. and Larson, C.L. 1979. Development and Evaluation of Infiltration Model Parameters. ASAE & CSAE Summer Meeting, Paper 79-2045.

Wischmeier, W.H. and Smith, D.D. 1965. Predicting Rainfall-Erosion Losses from Cropland East of the Rocky Mountains. Agric. Handbook 282, 47 p.

Wischmeier, W.H. and Smith, D.D. 1978. Predicting Rainfall Erosion Losses - A Guide to Conservation Planning, USDA Handbook 537.

Yang, C.T. and Stall, J.B. 1976. Applicability of Unit Stream Power Equation. Proc. of ASCE, J. of Hydraulics Division, Vol. 102(HY5):559-568.

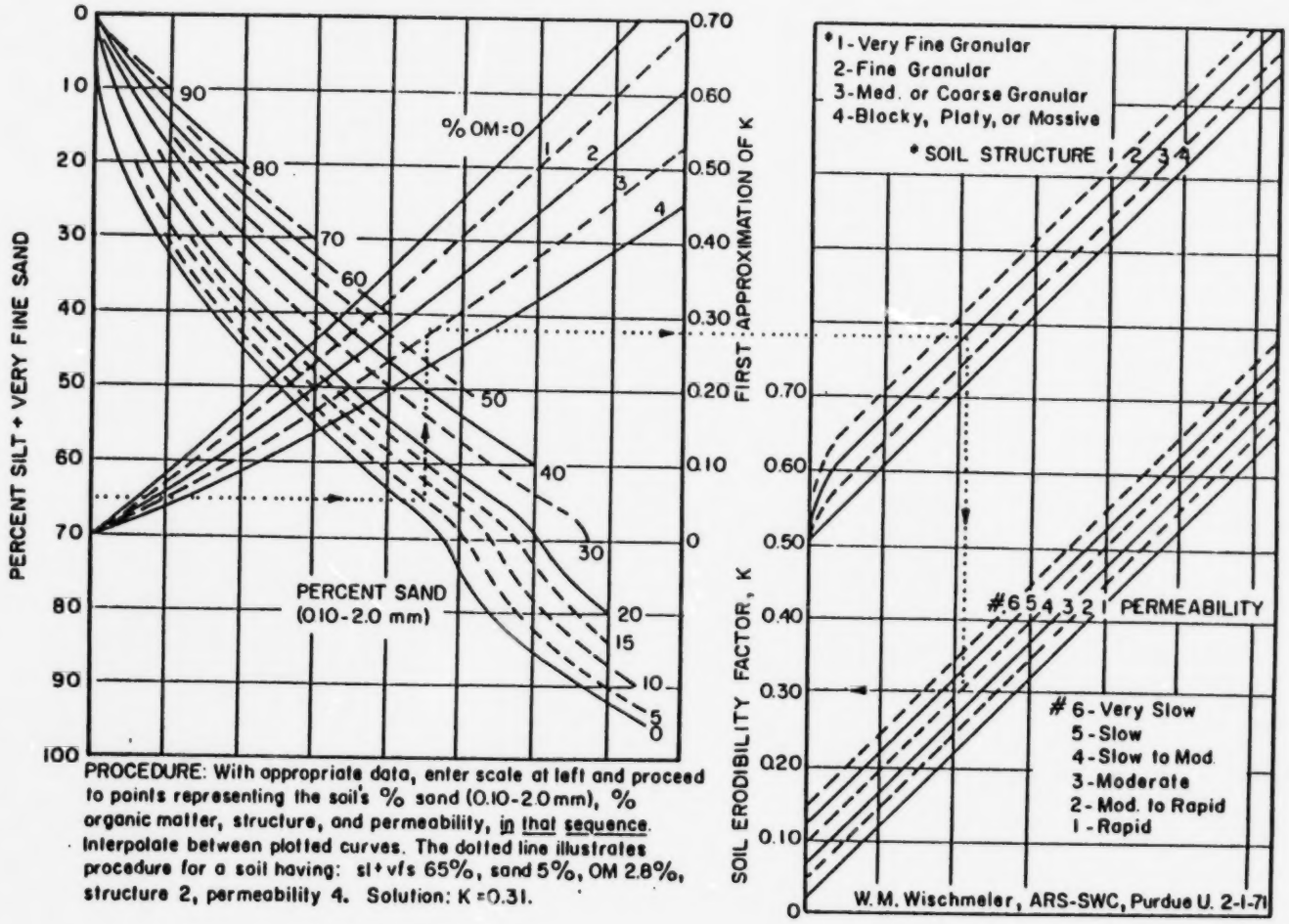
Youngberg, C.T. 1959. The Influence of Soil Conditions Following Tractor Logging on the Growth of Planted Douglas-Fir Seedings. Soil Sci.Soc. Amer. J. 23:76-78.

Youngs, E.G. 1968. An Estimation of Sorptivity for Infiltration Studies from Moisture Movement Considerations. Soil Sci. 106: 157-163.

Zaki, H. and Leliavsky, S. 1948. Tail Erosion as a Factor Affecting the Safety Coefficient Against Piping, Proc. of the 3rd ICOLD, Vol. II, Questions No. 10, R. 4, 10 pp.

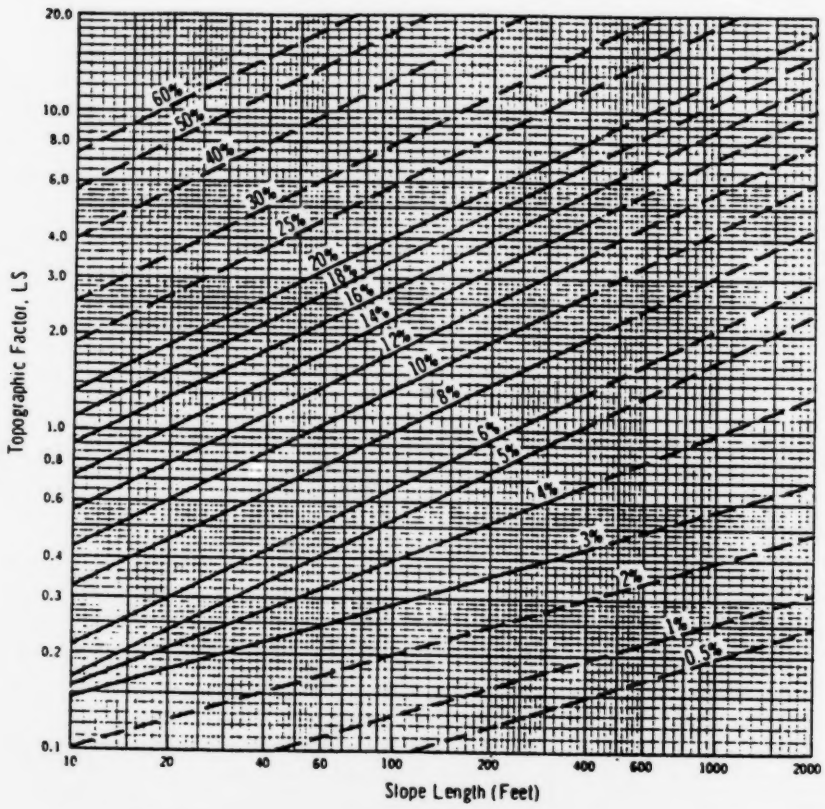
APPENDIX III.2.1

CHARTS AND TABLES FOR ESTIMATION OF UNIVERSAL SOIL LOSS EQUATION PARAMETERS



PROCEDURE: With appropriate data, enter scale at left and proceed to points representing the soil's % sand (0.10-2.0 mm), % organic matter, structure, and permeability, in that sequence. Interpolate between plotted curves. The dotted line illustrates procedure for a soil having: si+vfs 65%, sand 5%, OM 2.8%, structure 2, permeability 4. Solution: K = 0.31.

Nomograph for determining soil erodibility factor K (after Wischmeier, 1971).



*The dashed lines represent estimates for slope dimensions beyond the range of lengths and steepnesses for which data are available. The curves were derived by the formula:

$$LS = \left(\frac{\lambda}{72.6}\right)^{m} \left(\frac{430x^2 + 30x + 0.43}{6.613}\right)$$

where λ = field slope length in feet and m = 0.5 if $s \geq 5\%$ or greater, 0.4 if $s = 4\%$, and 0.3 if $s \leq 3\%$ or less; and $x = \sin\theta$. θ is the angle of slope in degrees.

Slope-effect chart (topographic factor, LS). (SCS, 1977)

Table Typical C Factor Values Reported in the Literature.*

	Condition	C factor
1.	Bare soil conditions	
	freshly disked to 6-8 inches	1.00
	after one rain	0.89
	loose to 12 inches smooth	0.90
	loose to 12 inches rough	0.80
	compacted root raked	1.20
	compacted bulldozer scraped across slope	1.20
	same except root raked across	0.90
	rough irregular tracked all directions	0.90
	seed and fertilize, fresh, unprepared seedbed	0.64
	same after six months	0.54
	seed, fertilize and 12 months chemical	0.38
	not tilled algae crusted	0.01
	tilled algae crusted	0.02
	undisturbed except scraped	0.66 - 1.30
	scarified only	0.76 - 1.31
	sawdust 2 inches deep, disked in	0.61
2.	Asphalt emulsion	
	1210 gallons/acre	0.01 - 0.019
	605 gallons/acre	0.14 - 0.57
	302 gallons/acre	0.28 - 0.60
3.	Dust binder	
	605 gallons/acre	1.05
	1210 gallons/acre	0.29 - 0.78
4.	Other chemicals	
	1000 lb fiber glass roving with 60-150 gallons/acre	0.01 - 0.05
	Aquatain	0.68
	Aerospray 70, 10 percent cover	0.94
	Curasol AE	0.30 - 0.48
	PVA	0.71 - 0.90
	Terra-Tack	0.66
	wood fiber slurry, 1400 lb/acre fresh	0.01 - 0.02
	wood fiber slurry, 3500 lb/acre fresh	0.10
5.	Seedings[1]	
	temporary, 0 to 60 days[2]	0.40
	temporary, after 60 days	0.05
	permanent, 2 to 12 months	0.05
6.	Brush	0.35

1. If plantings are used with mulches, use the minimum C values.
2. If dry weather occurs at planting and emergence is a problem, extend the 0-60 days to a period when rainfall normally occurs.

* National Cooperative Highway Research Program, 1976.

Table C Factors for Mechanically Prepared Woodland Sites.[4]

Percent of soil covered with residue in contact with soil surface	Soil Condition and Weed Cover							
	Excellent		Good		Fair		Poor	
	NC	WC	NC	WC	NC	WC	NC	WC
None								
A. Disked, raked or bedded[1,2]	.52	.20	.72	.27	.85	.32	.94	.36
B. Burned[3]	.25	.10	.26	.10	.31	.12	.45	.17
C. Drum chopped[3]	.16	.07	.17	.07	.20	.08	.29	.11
10% Cover								
A. Disked, raked or bedded[1,2]	.33	.15	.46	.20	.54	.24	.60	.26
B. Burned[3]	.23	.10	.24	.10	.26	.11	.36	.16
C. Drum chopped[3]	.15	.07	.16	.07	.17	.08	.23	.10
20% Cover								
A. Disked, raked or bedded[1,2]	.24	.12	.34	.17	.40	.20	.44	.22
B. Burned[3]	.19	.10	.19	.10	.21	.11	.27	.14
C. Drum chopped[3]	.12	.06	.12	.06	.14	.07	.18	.09
40% Cover								
A. Disked, raked or bedded[1,2]	.17	.11	.23	.14	.27	.17	.30	.19
B. Burned[3]	.14	.09	.14	.09	.15	.09	.17	.11
C. Drum chopped[3]	.09	.06	.09	.06	.10	.06	.11	.07
60% Cover								
A. Disked, raked or bedded[1,2]	.11	.08	.15	.11	.18	.14	.20	.15
B. Burned[3]	.08	.06	.09	.07	.10	.08	.11	.08
C. Drum chopped[3]	.06	.05	.06	.05	.07	.05	.07	.05
80% Cover								
A. Disked, raked or bedded[1,2]	.05	.04	.07	.06	.09	.08	.10	.09
B. Burned[3]	.04	.04	.05	.04	.05	.04	.06	.05
C. Drum chopped[3]	.03	.03	.03	.03	.03	.03	.04	.04

1. Multiply A values by following values to account for surface roughness:

Very rough, major effect on runoff and sediment storage, depressions greater than 6 inches	.40
Moderate	.65
Smooth, minor surface sediment storage, depressions less than 2 inches	.90

2. The C values for A are for the first year following treatment. For A type sites 1 to 4 years old multiply C value by .7 to account for aging. For sites 4 to 8 years old use Table 5.6. For sites more than 8 years old use Table 5.7.

3. The C values for B and C areas are for the first 3 years following treatment. For sites treated 3 to 8 years ago use Table 5.6. For sites treated more than 8 years ago use Table 5.7.

4. Soil Conservation Service (1977).

Table C Factors for Permanent Pasture, Rangeland, Idle Land, and Grazed Woodland.[1,5]

Vegetal Canopy			Cover that Contacts the Surface					
Type and Height of Raised Canopy[2]	Canopy Cover[3]	Type[4]	Percent Ground Cover					
	%		0	20	40	60	80	95-100
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.090	.043	.011
Canopy of tall weeds or short brush (0.5 m fall ht.)	25	G	.36	.17	.09	.038	.012	.003
		W	.36	.20	.13	.082	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.075	.039	.011
	75	G	.17	.10	.06	.031	.011	.003
		W	.17	.12	.09	.067	.038	.011
Appreciable brush or bushes (2 m fall ht.)	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.085	.042	.011
	50	G	.34	.16	.085	.038	.012	.003
		W	.34	.19	.13	.081	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.077	.040	.011
Trees but no appreciable low brush (4 m fall ht.)	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.087	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.085	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.083	.041	.011

1. All values shown assume: (1) random distribution of mulch or vegetation, and (2) mulch of appreciable depth where it exists. Idle land refers to land with undisturbed profiles for at least a period of three consecutive years. Also to be used for burned forest land and forest land that has been harvested less than three years ago.
2. Average fall height of waterdrops from canopy to soil surface: m = meters.
3. Portion of total area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).
4. G: Cover at surface is grass, grasslike plants, decaying compacted duff or litter at least 2 inches deep.

   W: Cover at surface is mostly broadleaf herbaceous plants (as weeds with little lateral root network near the surface), and/or undecayed residue.
5. Soil Conservation Service (1977).

APPENDIX III.6.1

APPLICATIONS OF DARCY'S LAW TO INFILTRATION ESTIMATION

APPENDIX III.6.1 APPLICATIONS OF DARCY'S LAW TO INFILTRATION ESTIMATION

A range of application of Darcy's law may be defined in terms of a Reynolds number for flow through a porous media. The Reynolds number for flow through a porous media is defined as:

$$Re = qd/\nu \qquad \text{(A.6.1.1)}$$

where d is a representative dimension of the grain sizes in the porous media, and is the kinematic viscosity of the fluid. The value d is usually represented by D_{10} or D_{50} for the grain sizes, although Collins (1961) suggests relating d to the permeability and porosity of the soil:

$$d = (k/n)^{1/2} \qquad \text{(A.6.1.2)}$$

where n is the porosity and k the intrinsic permeability.

Based on the work of Rose (1945), it has been suggested by Bear (1972) that flow through porous media may be divided into three distinct regions. Figure A.III.6.1-1 shows a plot of Fanning friction factor and Reynold's number. The Fanning friction factor is determined as:

$$f = 0.5d\,(\Delta\Psi/L)\,(\nu U^2) \qquad \text{(A.6.1.3)}$$

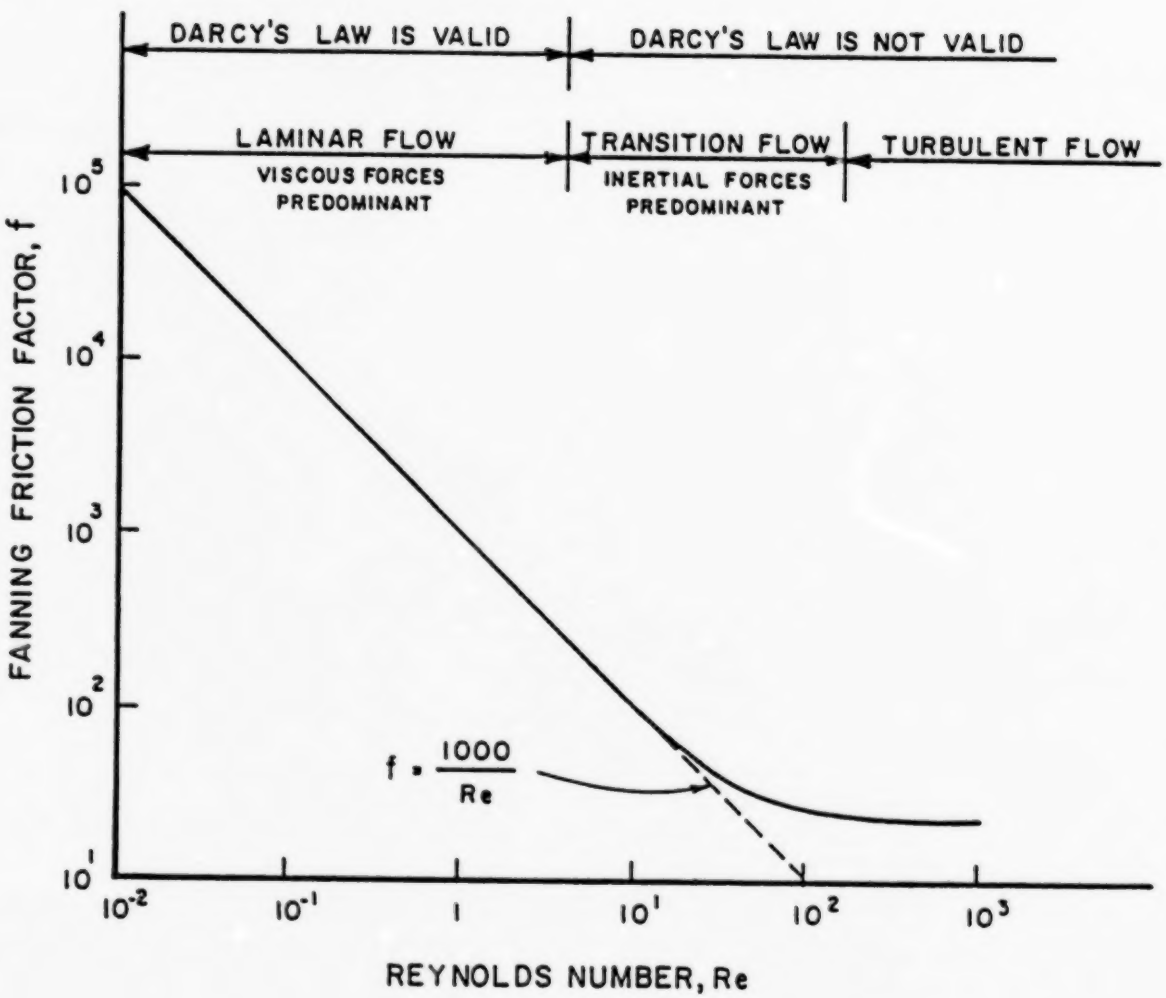
where $\Delta\Psi$ is the head drop over a length L in the direction of flow and V is the velocity of the flow. Based on the relationships presented in Figure A.III.6.1-1, it can be seen that Darcy's law is valid for values of the Reynolds number less than some value between 1 and 10. An average grain size, D_{50}, was used in this analysis.

The theoretical particle differential equation for unsaturated flow is obtained by combining Darcy's equation with the continuity equation. This equation is referred to as the diffusion equation or Richards equation (Swartzendruber, 1969). The equation is expressed as:

$$\frac{\partial\theta}{\partial t} = \frac{\partial}{\partial}\left[\left(D(\theta)\,\frac{\partial\theta}{\partial z}\right)\right] - \frac{\partial K(\theta)}{\partial z} \qquad \text{(A.6.1.4)}$$

where $D(\theta)$, the soil water diffusivity, is defined as $K(\psi)dZ/d\theta$. The difficulty with solving the equation is the nonlinear relationship between hydraulic conductivity, pressure head, and soil moisture. The equation can be expressed either with a θ base or with a Z base, and Skaggs and Khaleel(1979) presents a discussion on the selection of the equation base.

Several solutions of the equation can be developed based on the boundary conditions for the system being evaluated. For a saturated soil $d\theta/dt$ is zero, $K(\theta)$ reaches a constant, and the pressure head becomes positive. Equation A.6.1.4 reduces to the Laplace equation for one-dimensional



FANNING FRICTION FACTOR VS REYNOLDS NUMBER (BEAR, 1972)

FIGURE A.III.6.1-1

saturated flow. A discussion of solutions to Richards equation is presented in Appendix III.6.2. Three-dimensional saturated flow in an isotropic soil is expressed by the following form of the Laplace equation (Hillel, 1971):

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad \text{(A.6.1.5)}$$

where ϕ is the total potential head. The total potential head is the sum of the pressure head and the gravity potential above the datum plane.

APPENDIX III.6.2

RICHARDS' EQUATION

APPENDIX III.6.2 RICHARDS' EQUATION

A computer model described by Moore and Eigel (1981) was used in this study to provide a finite difference solution of the one-dimensional form of Richards' equation for a non-swelling soil (Smith and Woolhiser, 1971):

$$\frac{\partial\theta}{\partial t}=\frac{\partial}{\partial z}\left[K_s k_r(\psi)\frac{\partial\psi}{\partial z}\right]-\frac{\partial[K_s k_r(\psi)]}{\partial z} \qquad \text{(A.6.2.1)}$$

where θ = volumetric moisture content,

K_s = saturated hydraulic conductivity,

$k_r(\psi)$ = relative hydraulic conductivity,

ψ = pressure head,

z = distance below the surface, and

t = time

A more convenient form of equation (A.6.2.1) is:

$$c\,\frac{\partial\psi}{\partial t}=\frac{\partial}{\partial z}\left[K_s k_r(\psi)\frac{\partial\psi}{\partial z}\right]-\frac{\partial[K_s k_r(\psi)]}{\partial z} \qquad \text{(A.6.2.2)}$$

where c is the moisture capacitance $\partial\theta/\partial\psi$. This equation has no exact general analytical solution. The equation is a second-order, non-linear partial differential equation for unsaturated flow in a porous media where air moves under negligible pressure gradients. An implicit Crank-Nicolson finite difference scheme was used to provide a solution to equation (A.6.2.2). In the next section, a step-by-step discussion of the procedures used in the computer model is presented.

Crank-Nicolson Scheme

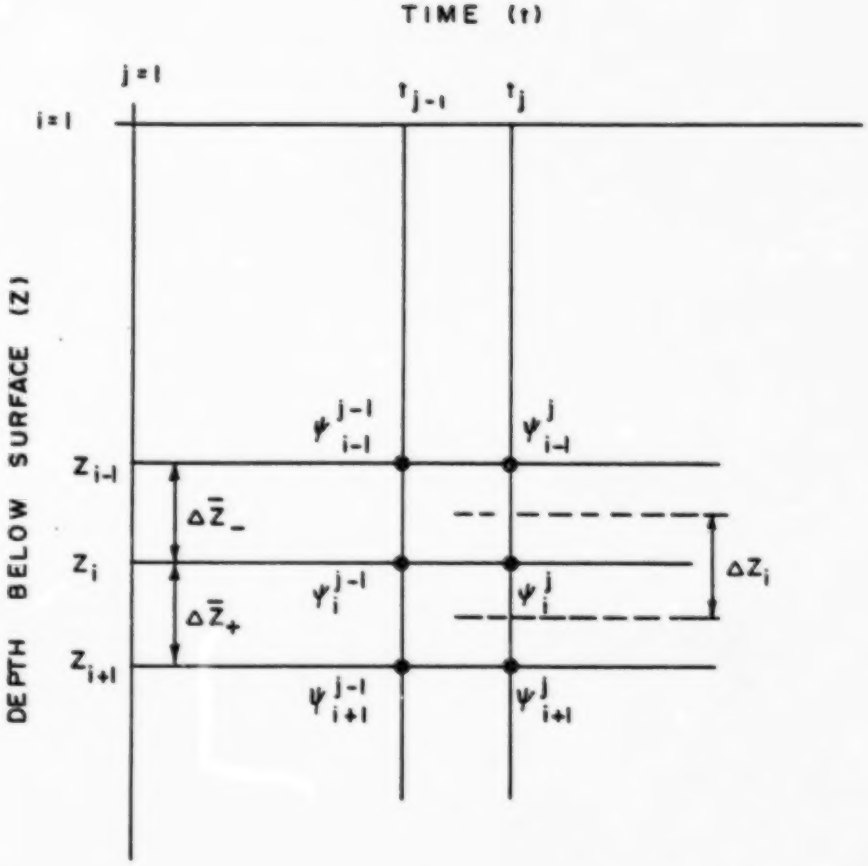
The general form of the Crank-Nicolson formula can be written as (Mitchell, 1969):

$$(1+r)U_m^{n+1}-\frac{1}{2}r\left[U_{m+1}^{n+1}+U_{m-1}^{n+1}\right]=$$

$$(1-r)U_m^n+\frac{1}{2}r\left[U_{m+1}^n+U_{m-1}^n\right] \qquad \text{(A.6.2.3)}$$

where $r=k/h^2$

For an equation varying in time and space, the grid parameters k and h represent Δt and Δz, respectively. The scheme converges for $r > 0$. In the scheme used in the computer model, a variable grid size system is used and reference should be made to Figure A.III.6.2-1. The finite difference form of equation (A.6.2.2) written for node (i,j) is:



GRID NETWORK FOR FINITE DIFFERENCE SCHEME
(SOURCE: MOORE AND EIGEL, 1981)

FIGURE A.III.6.2-1

$$c_i^{j-1/2}\left[\frac{\psi_i^j-\psi_i^{j-1}}{\Delta t^{j-1}}\right]\Delta z_i =$$

$$K_{s+}K_{r+}^{j-1/2}\left[\frac{\psi_{i+1}^j-\psi_i^j+\psi_{i+1}^{j-1}-\psi_i^{j-1}}{2\Delta\bar{z}_+}-1\right]- \qquad \text{(A.6.2.4)}$$

$$K_{s-}K_{r-}^{j-1/2}\left[\frac{\psi_i^j-\psi_{i-1}^j+\psi_i^{j-1}-\psi_{i-1}^{j-1}}{2\Delta\bar{z}_-}-1\right]$$

$$\text{where } K_{s+}=\frac{2\Delta\bar{z}_+}{\frac{\Delta z_{i+1}}{K_{s_{i+1}}}+\frac{\Delta z_i}{K_{s_i}}} \quad ; \quad K_{s-}=\frac{2\Delta\bar{z}_-}{\frac{\Delta z_i}{K_{s_i}}+\frac{\Delta z_{i-1}}{K_{s_{i-1}}}}$$

and $K_r^{J-1/2}$ and $c^{j-1/2}$ are evaluated at each iteration from the estimated pressure head midway between the j and j - 1 time levels. The other terms are defined in Figure A.III.6.2-1. Equation (A.6.2.4) can be rearranged and written in terms of the unknown variables ψ_{i-1} ψ and ψ_{i+1} as:

$$A_i\psi_{i-1}^j+B_i\psi_i^j+C_i\psi_{i+1}^j=D_i \qquad \text{(A.6.2.5)}$$

where A, B, C and D are coefficients calculated at each iteration from the known values in equation (A.6.2.4). An equation of the form of equation (A.6.2.5) can be written for the upper and lower boundary conditions and for each interlying depth mode, resulting in a set of N simultaneous equations in N unknowns where there are N node points in place. The N simultaneous equations form an N x N tri-diagonal matrix which can be solved by a simple numerical algorithm.

Upper Boundary Conditions

Two conditions might occur at the surface; either the surface is saturated or the surface is unsaturated. If the surface is saturated, the capillary suction, ψ_1^j is set to zero (pond is assumed negligible), and the computations are started at node 2.

If the surface is unsaturated, the rainfall rate is added to equation (A.6.2.4) and for node 1, the equation becomes:

$$K_{s+} k_{r+}^{j-1/2} \left[\frac{\psi_2^j - \psi_1^j + \psi_2^{j-1} - \psi_1^{j-1}}{2\Delta z_-} - 1 \right] + I^j$$

$$= c_1^{j-1/2} \left[\frac{\psi_1^j - \psi_1^{j-1}}{\Delta t^{j-1}} \right] \Delta z_1 \qquad \text{(A.6.2.6)}$$

where I^j is the specified rainfall rate at time t^j.

Lower Boundary Conditions

In the early stages of infiltration, the wetting front does not penetrate far into the profiles. To make the computer model more economic, equation (A.6.2.5) is truncated by excluding some of the lower nodes which have not been reached by the wetting front. Equation (A.6.2.5) becomes:

$$A_m \psi_{m-1}^j + B_m \psi_m^j = D_m \qquad \text{(A.6.2.7)}$$

where equation (A.6.2.7) is the final equation of the truncated tri-diagonal m x m matrix. At the node m, $\psi_m^j = \psi_m^{j-1}$. As the wetting front approaches the lower boundary of the truncated matrix, the matrix is redefined until it encompasses all the nodes in the original N x N matrix. The lower boundary is then specified by:

$$\psi_N = \psi_N^{j-1}$$

Initial Conditions and Soil Properties

The depth increments between the nodes must be specified. The computer model permits the input of variable depth increments. At the soil surface and at the interface between nonhomogeneous layers, the depth increment should be made small. At nodes where the suction gradients are much lower, the depth increments can be made larger. The depth increments used in this study are shown in Table A.III.6.2-1.

Initial soil moisture content must be specified at each node. To determine a solution for equation (A.6.2.7) the soil characteristic curve and the hydraulic conductivity-suction relationship for each node must also be specified. These two relationships may be entered as tabular data or defined by a series of Campbell's equations for each soil type. If Campbell's equation is used, then the 'x', 'b', and K_s (or K_{fs}) values for each soil type are entered together with a specification of which nodes are associated with each soil type. This approach was used in this study.

To prevent instability near saturation, the model uses the Clapp and Hornberger (1978) correction method for the soil characteristics curve. An "arbitrary' inflection point near saturation must be specified. This point is selected so that the suction at the inflection point is close to the air entry suction value. When the degree of saturation of the soil is greater

TABLE A.III.6.2-1

FINITE DIFFERENCE SCHEME DEPTH INCREMENTS

Depth From Surface (cm)	Depth Increment Size		
	Soil Profile (cm)	Topsoil/Spoil Profile (cm)	Topsoil/Soil Profile(1) (cm)
0.00- 0.05			0.01
0.05- 0.30			0.05
0.30- 1.30			0.10
1.30- 3.80			0.50
3.80-13.80			1.00
13.80-28.80			1.50
28.80-38.80			2.00
0.00- 1.00	0.10	0.10	
1.00- 3.00	0.20	0.20	
3.00- 5.50	0.50	0.50	
5.50-15.50	1.00	1.00	
15.50-30.50	1.50	1.50	
30.50-40.50	2.00	2.00	

(1) Surface sealing

than that at the inflection point, the soil characteristic curve is defined by the parabolic equation:

$$\Psi = -m(s-n)(s-1) \qquad \text{(A.6.2.8)}$$

where s is the degree of saturation (θ/θ_s). The parameters m and n are calculated from the equations:

$$m = \frac{\Psi_{in}}{(1. - s_{in})^2} - \frac{\Psi_{in}b}{s_{in}(1 - s_{in})} \qquad \text{(A.6.2.9)}$$

$$\text{and} \quad n = 2s_{in} - [\Psi_{in}b/ms_{in}] - 1 \qquad \text{(A.6.2.10)}$$

where Ψ_{in} and S_{in} are the capillary suction and the degree of saturation at the inflection point.

Iterative Scheme

To solve equation (A.6.2.4) at a given time step, values of K_r and c must be determined for each node. However, to obtain these values, corresponding values of Ψ need to be known. To develop a solution, therefore, an iterative scheme must be used. To accelerate convergence to within a desired error tolerance the Aitken method is incorporated in the computer model. The formula for the method can be expressed as:

$$\Psi_{n+2} = \Psi_n - \frac{(\Psi_{n+1} - \Psi_n)^2}{(\Psi_{n+2} - 2\Psi_{n+1} + \Psi_n)} \qquad \text{(A.6.2.11)}$$

where Ψ_n, Ψ_{n+1}, and Ψ_{n+2} are the values of Ψ from the previous three iterations.

Infiltration Determination and Time Step adjustment

After convergence is achieved, the total infiltration volume from the beginning of an event in question is calculated together with the average infiltration rate for the last time step. If convergence is not achieved within a specified number of iterations, the time step is reduced and the iterative procedure repeated. If convergence is still not achieved, the iterative procedure is repeated using the minimum specified time step. If convergence is still not achieved, the computer model terminates the solution within an error message. A reduction of 20 percent in the time step is made if the initial number of interations exceeded ten.

At the end of each time step, a new time increment is calculated based on the number of iterations at the previous time step and the slope change in the suction gradient at each node over the previous time step. The time step was increased by 20 percent if the number of iterations in the previous time step was less than four and was increased by less than 20 percent if the number of iterations was greater than four. Depending on the slope changes, the time step could be reduced to a time increment of not less than the minimum specified time increment. The model also includes

options to simulate initial saturation at the surface or the formation of a transient seal in the surface layer. These options, however, were not used in this study.

APPENDIX III.6.3

GREEN-AMPT PARAMETER ESTIMATION

APPENDIX III.6.3 GREEN-AMPT PARAMETER ESTIMATION

In 1966 Bouwers developed a procedure for measuring the air entry value and hydraulic conductivity of a soil. Based on a review of the work by Corey et al. (1965) he assumed that the hydraulic conductivity in the wetting zone was 0.5 K_s and the pressure above the wetting front was 0.5 ψ_a, where ψ_a is the air entry value of soil. Bouwers' experimental results substantiated these selections of the hydraulic conductivity and pressure at the wetting front.

Brakensiek and Onstad (1977) found that the effective capillary pressure at the wetting front can be estimated as 0.76 times the desorption bubbling pressure*. The work of Brakensiek was based on an equation developed by Morel-Seytoux and Khanji (1974). They developed the equation:

$$S_w = \int_o^{\psi_i} K_r d\psi \qquad \text{(A.6.3.1)}$$

where K_r is the relative water conductivity (K/K_s), ψ is the capillary pressure, and ψ_i is the capillary pressure at initial soil moisture content. By combining this relationship with the procedures of Brooks and Corey (1964), they obtained the relationship:

$$S_w = \left[\frac{2+3\lambda}{1+3\lambda}\right] \psi_e \qquad \text{(A.6.3.2)}$$

where ψ_e is the air exit capillary pressure** and λ is the slope of the log-log plot of effective saturation as a function of capillary pressure for desorption. The desorption bubbling pressure is estimated as twice the air exit capillary pressure.

A statistical procedure was developed by Brakensiek and Onstad (1977) for estimating S_w and K using infiltration data. Brakensiek and Onstad report that the fillable porosity can be estimated by the relationship:

$$n = C(1-\gamma/2.65) - \theta_i \qquad \text{(A.6.3.3)}$$

---

* The minimum capillary pressure at which the air permeability is greater than zero.

**The capillary pressure at which air exits the soil during sorption.

where θ_i is the antecedent soil moisture, γ is the bulk density, and C is a factor used to reduce total porosity. In a study by Onstad et al. (1973), a value of 0.86 was used for the factor C. Using regression techniques and the data of Nielsen et al. (1973), Brakensiek and Onstad developed the regression equation:

$$\psi^{0.55} = 54.375 - 112.736\theta \qquad \text{(A.6.3.4)}$$

where ψ is the soil water head in centimeters and θ is the average water content.

APPENDIX III.6.4

GREEN-AMPT-MEIN-LARSON MODEL (GAML)

APPENDIX III.6.4 GREEN-AMPT-MEIN-LARSON MODEL (GAML)

Green and Ampt (1911) developed an infiltration equation for ponded surfaces based on Darcy's Law and a capillary-tube analogy. This equation can be written as:

$$f = K_s[(L + S)/L] \qquad \text{(A.6.4.1)}$$

where S is the capillary suction at the wetting front, L is the depth to the wetting front from the surface, K_s is the saturated hydraulic conductivity of the wetted zone, and f is the infiltration rate. If K_s is redefined as K_{fs} and the infiltration volume, F, from continuity, is made equal to $L(\theta_{fs} - \theta_i)$, equation (A.6.4.1) can be written as:

$$f = \frac{dF}{dt} = K_{fs}(1 + (\theta_{fs} - \theta_i)S/F) \qquad \text{(A.6.4.2)}$$

The depth of surface ponding has been assumed negligible in equations (A.6.4.1) and (A.6.4.2). If equation (A.6.4.2) is integrated, the equation becomes:

$$K_{fs}t = F - S(\theta_{fs} - \theta_i)Ln[1 + F/S(\theta_{fs} - \theta_i)] \qquad \text{(A.6.4.3)}$$

where t is the time after infiltration begins. Derivation of equation (A.6.4.3) assumes surface ponding through the infiltration process.

Mein and Larson (1961) modified the Green-Ampt model to account for infiltration prior to surface ponding. Their two-stage infiltration model is described by two equations. Stage 1, up to the time of surface ponding, t_s, is described by:

$$F_s = \frac{S(\theta_{fs} - \theta_i)}{[(I/K_{fs}) - 1]} \qquad \text{(A.6.4.4)}$$

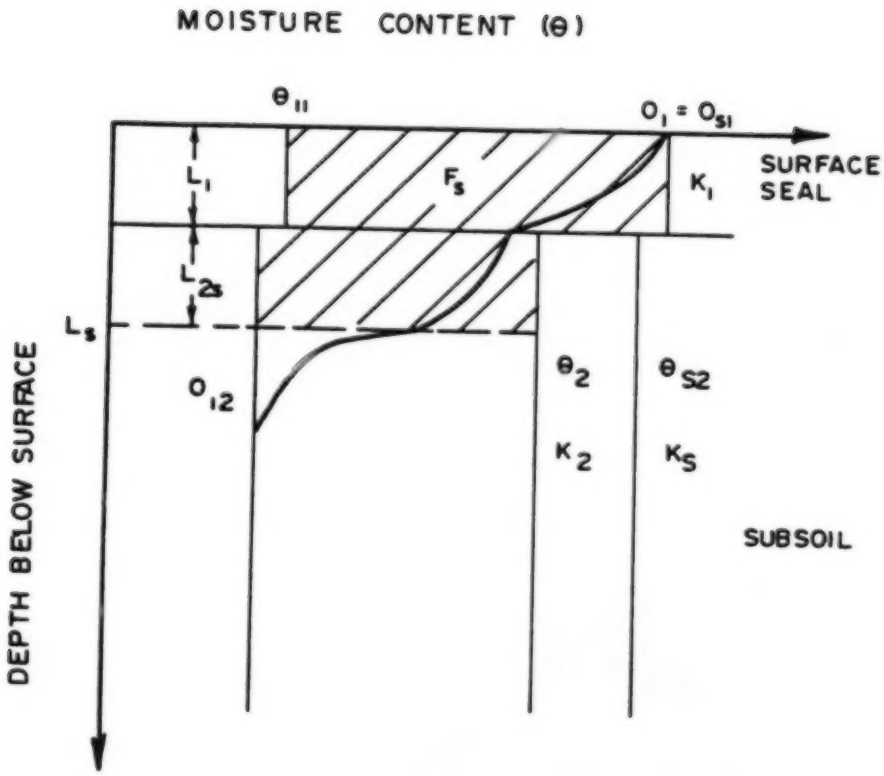
where F_s is the volume of infiltration at the time of surface ponding. At the time of surface ponding, the infiltration rate is equal to the rainfall rate and $t_s = F_s/I$. The second stage of infiltration is determined using a modified form of equation (A.6.4.3):

$$K_{fs}(t - t_s + t_s') = F - S(\theta_{fs} - \theta_i)Ln[1 + F/S(\theta_{fs} - \theta_i)] \qquad \text{(A.6.4.5)}$$

where t_s', is the time required to infiltrate a volume equivalent to F_s under ponded surface conditions. The solution for t_s is obtained by substituting for F_s in equation (A.6.4.3) and solving for t.

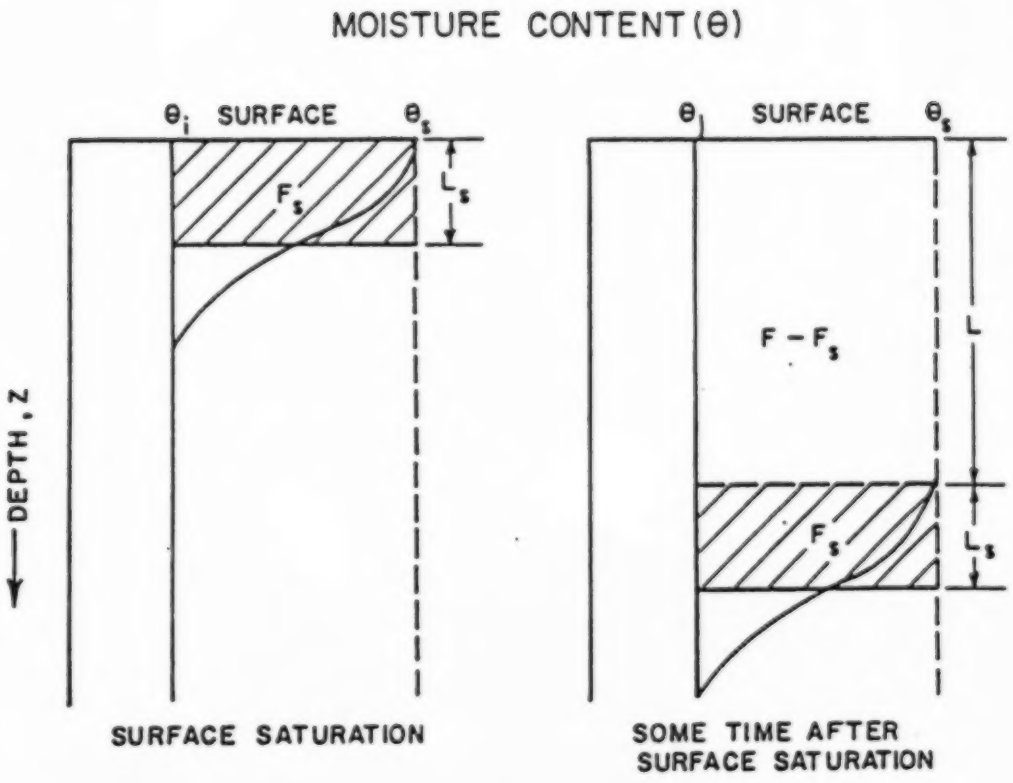
Moore (1981) developed a solution of the GAML model for a two-layer soil profile. The conceptual soil profiles for the procedure are illustrated in Figures A.III.6.4-1 and A.III.6.4-2. If Darcy's law is applied to the system and the depth of ponding is negligible, f is determined as:

$$f = \frac{L + S_{w2}}{\frac{L_1}{K_1} + \frac{L_2}{K_2}} \quad \text{for } L \geq L_1 \qquad \text{(A.6.4.6)}$$



CONCEPTUAL MOISTURE PROFILE AT MOMENT OF SURFACE PONDING (SOURCE: MOORE, 1981)

FIGURE A.III.6.4-1



GENERALIZED SOIL MOISTURE PROFILES DURING INFILTRATION AT (A) THE MOMENT OF SURFACE SATURATION AND (B) AT A LATER TIME

FIGURE A.III.6.4-2

where S_{w2} is the capillary drive at the wetting front in the subsurface soil. At time, t, the volume of water infiltrated is:

$$F = L_1 \Delta\theta_1 + L_2 \Delta\theta_2 \qquad \text{(A.6.4.7)}$$

By substituting f = dF/dt into equation (A.6.4.6) and combining equations (A.6.4.6) and (A.6.4.7) the following expression for dF/dt is obtained:

$$\frac{dF}{dt} = \frac{L_1 + \dfrac{F - L_1\Delta\theta_1}{\Delta\theta_2} + S_{w2}}{\dfrac{L_1}{K_1} + \dfrac{F - L_1\Delta\theta_1}{\Delta\theta_2 K_2}} \qquad \text{(A.6.4.8)}$$

If equation (A.6.4.8) is integrated between the limits t = t_1 (F=F1) and t = t, the following expression for the infiltration process is obtained:

$$F + (E - H)Ln\left[1 + \frac{F - F_1}{H}\right] = K_2(t - t_1) + F_1 \qquad \text{(A.6.4.9)}$$

where $E = L_1 \theta_2 (K_2/K_1)$; $H = \theta_2 (L + S_{w2})$; and $F = L_1\theta_1$. Moore (1981) notes that equation (A.6.4.9) was first presented by Van Duin (1955) and was later presented with a complete derivation by Asseed and Swartzendruber (1974).

Equation (A.6.4.9) is the form of the Green-Ampt equation for single stage infiltration through a two-layer system. For the GAML model for two-stage infiltration through a two-layer system, the equation can be written in the same form as equation (A.6.4.6).

$$F + (E - H)Ln\left[1 + \frac{F}{(H - F_1)}\right] = K_2(t - t_s + t_s') \qquad \text{(A.6.4.10)}$$

where F_s is given by the equation:

$$F_s = \frac{H - E\,\dfrac{I}{K_2}}{\dfrac{I}{K_2} - 1} + F_1 \qquad \text{(A.6.4.11)}$$

A complete derivation of equation (A.6.4.11) is presented by Moore (1981). Equation (A.6.4.11) applies if F_s exceeds the storage volume in the surface layer. If $L_s > L_1$ equation (A.6.4.4) is used with parameter estimates for the surface layer. The procedure is well suited to solution by computer and the model developed by Moore (1981) was used in this study.

Parameter Determination for the Modified GAML Procedure

Use of the procedure requires knowledge of the relationship between the hydraulic conductivity and capillary suction, the soil water characteristic relationship, and the porosity and depth of each layer.

Based on the excellent results obtained with the Campbell Method, the suction at the wetting front can be determined by a procedure presented by Moore (1980b):

$$S_w = \frac{\int_{K_{ri}}^{K_{rf}} S(K_r)\,dK_r}{K_{rf} - K_{ri}} \qquad \text{(A.6.4.12)}$$

Where K_{rf} is the relative hydraulic conductivity at field saturation (K/K_s at θ_{fs}) and K_{ri} is the relative hydraulic conductivity at the initial moisture content θ_i. The relative hydraulic conductivity can be calculated by using the Campbell equation:

$$K_{rj} = (\theta_j / \theta_{fs})^{2b+3} \qquad \text{(A.6.4.13)}$$

Where K_{rj} is the relative conductivity at the moisture content θ_j and b is obtained from the relationship:

$$\psi_j = x\,\theta_j^{-b} \qquad \text{(A.6.4.14)}$$

Where ψ_j is the suction at moisture content θ_j and x and b are constants for a particular profile. If equation (A.6.4.13) is substituted into equation (A.6.4.12) the following solution for S_w is obtained:

$$S_w = \frac{\psi_e\,(K_{rf}^a - K_{ri}^a)}{a(K_{rf} - K_{ri})} \qquad \text{(A.6.4.15)}$$

where $a = (b + 3)/(2b + 3)$ and ψ_e is the air entry water potential.

The approach is not restricted to a two layer system and can easily be modified to account for more layers.

APPENDIX III.6.5

NUMERICAL SOLUTIONS TO THE FLOW EQUATIONS

APPENDIX III.6.5 NUMERICAL SOLUTIONS TO THE FLOW EQUATIONS

In section III.6.4, several simple approximate solutions to the flow equations are presented. In recent years implicit and explicit finite difference solutions of the Richards equation have been developed for application in computer models. It should be noted, however, that although these solutions might more accurately describe the flow process through a soil, their solution is still approximate. Errors in the solution result from the hysteretic nature of the soil properties, the inability to accurately determine the soil properties, the use of approximate numerical techniques to solve partial differential equations, and simple assumptions made in defining initial and boundary conditions of the solution. A good review of the numerical techniques frequently used to solve the Richards equation is presented by Skaggs and Khaleel (1979). Theoretical examples of the efffect of a surface seal on cumulative infiltration are shown in Figure A.III.6.5-1.

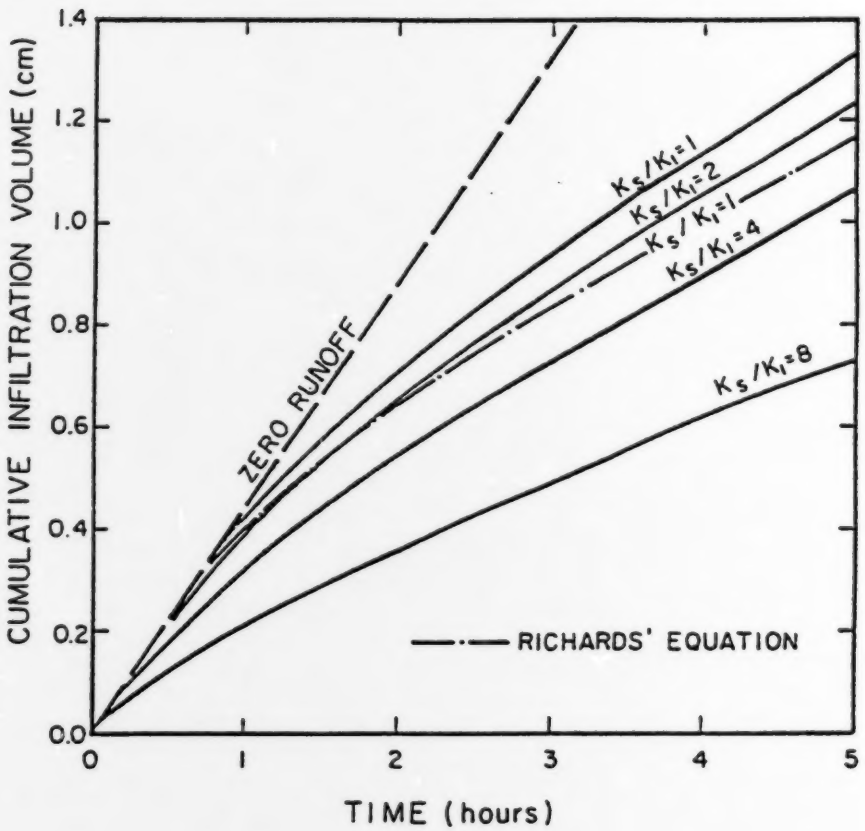


FIGURE III.6.5-1 CUMULATIVE INFILTRATION VOLUME VS TIME (I/K_2=4, θ_i=0.40) (SOURCE: MOORE, 1979a)